DEPARTMENT OF DEFENSE

PROTECTIVE CONSTRUCTION REVIEW GUIDE (HARDENING)

VOLUME I

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PROTECTIVE CONSTRUCTION REVIEW GUIDE--HARDENING

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FOREWORD

The term "Protective Construction" embraces those passive measures which can be effected by construction to nullify the effects of weapons and to enhance the recuperability of weapon systems and facilities. The term includes dispersion and duplication of services as well as strengthening of structures, camouflage and the incorporation of protection against chemical, biological and radiological agents. The term does not fully embrace all elements of passive defense, such as electronic countermeasures, control of electronic emissions, control of lighting and damage control.

"Hardening" is used to define that form of protective construction related to structural blast and shock resistance to the effects of nuclear weapons. A physical barrier, the hardened structure, is interposed between the weapon and the operating function to be protected.

Protection is provided by "sheltering" the personnel and equipment, or by dispersing the facilities which they required, or by building duplicate facilities. Although the end product is a military construction item, there are numerous factors other than engineering and construction which are basic to the problem and have major influence on the final solution.

This document provides a systematic procedure for integrating the essential strategic, operational and engineering factors which must be considered in developing a program of "hardening" from the inception of the requirement through the construction of the facility. The procedure used is summarized in the SYNOPSIS and is treated in detail in the various sections of this Guide.

Since hardening is a new field and since active studies and experiments are continually underway, it can be expected that any report summarizing the current state of information will necessarily be incomplete. Moreover, because many aspects of the subject have been insufficiently studied heretofore, it is necessary to make certain assumptions and to develop certain treatments using the best engineering judgment and experience that it is possible to bring to bear on the subject. Consequently it may appear that some of the recommendations and procedures given herein are based neither on analytical studies nor on direct experimental evidence. In these instances an attempt has been made to bring to bear knowledge of related fields in an effort to make available the best possible recommendations on the subject at the present time.

As further knowledge becomes available, it is expected that revisions to parts of this review guide will be made in order that the material in it can remain up-to-date and useful. It is hoped that the publication of this review guide will not impede current or future programs of research and study which are urgently required to furnish better and more authoritative information on many of the topics treated. Without the background of fundamental studies which have been carried on in the past the present volume would have been impossible to write. Without further studies in many of the fields, revisions in the direction of greater economy or greater assurance of success in the design of protective structures will not be possible.

This is a document primarily for internal use, within OSD, for reviewing protective construction programs and more detailed projects. It may be used by other DOD offices as approved by their appropriate headquarters.

SYNOPSIS

Procedures for the study, justification, and review of protective construction projects are described herein. A synopsis of the recommended procedure for a particular facility or operation follows:

Determine the Strategic Category of the facility or operation by referring to SECTION 1. Then determine the required degree of protection, in terms of design blast overpressure, by the procedure of SECTION 2, TARGET ANALYSIS. It may be that no protection is required. However, having established the need for and level of protection, if required, it is necessary then to specify the personnel and materiel to be protected and the physical space and utilities required in the facility, by reference to SECTION 3, OPERATIONAL CONCEPTS AND REQUIREMENTS. The survival criteria for the personnel and materiel to be protected are next determined by reference to SECTION 4, DAMAGE CRITERIA. After these steps have been accomplished, it is then possible to consider the design of the facility or structure.

It is necessary at this point to determine the type of structure required for the various parts of the facility under study. This involves consideration of what the structure must do in attenuating blast, thermal radiation, nuclear radiation, and earth shock. The analysis of the structure can then be accomplished and design proportions selected by the methods and charts of SECTION 5, DESIGN ANALYSIS. Finally, the cost of the structure so designed may be estimated by reference to the charts contained in SECTION 5, or by means of an approximate cost breakdown of the structure.

It should be remembered that at various points in the review procedure it may be determined that protective construction is either unnecessary or impractical. Conversely, as obviously impractical or impossible conditions are encountered, search for alternative operations or solutions may be made. There are many ways in which the problem of excessive vulnerability may be approached. The solution may lie in ingenious manipulation of location, alternative methods of operation, or in appropriate engineering design. One must consider all of these possibilities to insure the most economical solution.

Attention is called to certain basic premises which must be kept in mind in all cases:

a. The vulnerability of all elements essential to the operation must be considered. An otherwise adequate structural solution may be vulnerable to ground shock, for example; or a missile installation may be adequately protected in all respects except for the fuel lines connecting the fuel supply to the missile enclosure.

b. Alternate methods of protection should be considered. These include dispersal, duplication, alert status, as well as structural protection.

c. The method and level of protection should be consistent with the increment in survival probability which it produces.

d. Protection should be provided only to those elements which are essential to the operation, and even in these cases, space and other allowances and requirements should be reduced to a minimum.

e. The cost of physical protection is high and the need for more protection is extensive. Consequently, the more the unit cost of protection can be reasonably reduced, the more items can be protected.

f. The solution to difficult problems of protection often may lie in ingenious, unconventional concepts of operation and design. In most cases, direct application of conventional layouts, operational requirements, or design concepts will lead to impossible or impractical solutions.

An illustrative example of the procedures described is contained in SECTION 6. It may be noted that the procedures may be used both for preliminary design and for the cost review of a structure which has been designed. The methods and charts given herein are not intended to be used for final design or for accurate assessment of vulnerabilities. In many cases the methods of this review guide may be accurate enough for such purposes in view of the many uncertainties in the basic data; however, more precise engineering studies can be made by use of the reference material in SECTION 7, which also contains the acknowledgments.

This review guide is issued in two volumes. Volume I is unclassified, and contains all the material related to target analysis and to structural analysis and design, including all necessary charts. Volume II contains the material relating to strategic categories, operational concepts and requirements, and damage criteria, including some supplementary material which, because of security classification, could not be made available in Volume I. Copies of either volume can be obtained only through appropriate military agencies.

SECTION 1. See Volume II

SECTION 2. TARGET ANALYSIS

SECTION 2. TARGET ANALYSIS

2.1 INTRODUCTION

2.1.1 Purpose of Target Analysis. Target analysis is used to determine the level of hardness required in order for a facility to attain a desirable probability of survival against enemy attack. It is thus one of the several steps in protective construction planning. The planner analyzes his facility as though it were a target, using the best estimate available of enemy capabilities and intentions. From this analysis can be determined the extent of structural protection required to give a survival probability consistent with the importance of the facility.

2.1.2 Nature of Problem. Much of the work done in target analysis has been from an attack or offensive viewpoint. The weapons and delivery are the known quantities and the target properties are the estimated quantities. This is in effect the reverse of the present problem. In this problem the weapons and delivery must be estimated, and from this the target is constructed or hardened and is thus a "known" quantity. The element of probability lies principally in what weapon, delivery method, and time an enemy might choose.

There is still another important difference. In any analysis, inaccurately-known or estimated quantities are usually taken at the conservative boundary of the range of the possible values. This value, if conservative for one case, may be unconservative for the other case. Consequently, one must approach the defensive problem with a different viewpoint than in the offensive situation. When a choice of values is necessary, one should choose the value which is of greatest advantage to an enemy. This is of course the conservative choice from the viewpoint of protection construction.

Since the elements of weapon selection and delivery are not within the control of the planner, his only control over survival probability lies in strengthening his defenses. Aside from strengthening his air defenses, he can harden, duplicate, disperse, or employ a combination of these. His task is to obtain the desired survival expectancy for the least cost without sacrificing operational efficiency.

2.1.3 Procedures for Target Analysis. Two means of analyzing a facility as a possible target will be presented. Procedure A is the shorter and simpler procedure utilizing "Blast and Fallout Probability" charts prepared by a Department of Defense Hazard Study. This procedure is the simpler because most of the computations of target analysis have been performed. It should be used whenever possible. However, the use of Procedure A is limited to CONUS installations which are already in existence or sited.

The second procedure, called Procedure B, is really a collective designation for the several ways to be described of making an independent analysis of a target. Any of these ways is usually more laborious than

Procedure A, but in the case of an overseas installation or a CONUS installation which has not yet been sited, it will be necessary to resort to the use of the computations described in Procedure B. On the other hand, Procedure B is more useful as a design tool and is extremely flexible in its application.

2.2 DEFINITIONS AND NOTATION

2.2.1 Definitions. Key words and terms are defined as follows:

Target. The target is a structure, facility or installation which is not excessively large, so that it can be assumed without serious error to be subjected to uniform levels of overpressure and radiation from a given weapon.

Point Target. A target whose plan area is so small relative to the other distances involved in the analysis that the area of the target can be considered to be concentrated at a single point.

Hardness of a Structure. Peak side-on overpressure (psi) which the structure will tolerate, where side-on overpressure is the unreflected pressure in air at the surface of the ground at the structural site.

Vulnerability Radius. The radius in feet of a circle, having the target at its center, within which the detonation of a specific yield weapon will produce destruction of the target.

Vulnerability Circle. Circle formed by Vulnerability Radius.

Overlapping Targets. Targets having intersecting Vulnerability Circles.

Dependent Targets. Two or more targets whose spacing is such that each of them has a finite probability of being hit by any given attack weapon. Overlapping targets are dependent but targets need not be overlapping to be dependent.

Independent Targets. Two or more targets whose spacing is such that no more than one of them has a finite probability of being hit by any given attack weapon.

Designated Ground Zero. The ground location, for a surface burst at which the enemy will try to detonate his weapon, or for planning purposes, the location assumed to be the probable detonation point.

Circular Probable Error. It is assumed that delivery errors are equally probable in all directions from a point target. This results in a circular error described by the standard Gaussian Error Distribution. A measure of the error concentration is the CEP, which is the radius of a circle

about the DGZ within which one half of the weapons reaching the target area will fall. The CEP is largely a function of the method of weapon delivery.

Probability. The probabilities used in formulas are expressed as decimals. It is customary to speak of them as percent.

2.2.2 Notation.

W = Weapon yield in Megatons

CEP = Radius in feet of Circle of Equal Probability or Circular Probable Error

DGZ = Designated Ground Zero

n = Number of weapons launched at a target or complex

Z = Probability of an attack launched by an enemy reaching the target vicinity, i.e., performing mechanically and penetrating the friendly active defenses.

Z can be thought of as the probability of arrival. In the discussion and illustrations to follow Z is taken to be unity or 100%. If Z is not 100%, the hit probabilities must be multiplied by Z.

p_{so} = Peak side-on overpressure in air at the ground surface

R_v = Radius of Vulnerability in feet

A = Area of a complex or a non-point target

m = Number of equal targets

S = Survival probability computed

S' = Survival probability desired

N = Number of "squares" counted, multiplied by their value, on the plot of Cells of Equal Probability, Gaussian Error Distribution. N represents the single-shot hit probability for a given set of conditions.

H = Probability of an enemy success

h = Number of targets hit

pre-subscripts = Number of attacks

post-subscripts = Number of targets, or if the post-subscript is a capital letter it designates a specific target

2.3 PROCEDURE A, USING DOD HAZARD STUDIES

2.3.1 Establish the Desired Survival Probability. This is a function principally of the importance or strategic category of the facility in question. It will not be possible always to attain the desired survival probability. Thus the values given below are really the starting limits or desirable goals as initial guidance in target analysis. They do not represent firm DOD policy. In a specific case, the desirable or recommended survival probability will be considered and approved individually by coordinated review on the part of various DOD offices. The reference survival values by strategic category of facility are as follows:

Strategic Category Code	Desired Survival Probability (Pct)
A	80 - 95
B	60 - 80
C	40 - 60
D	10 - 40

2.3.2 Determine Levels of Structural Protection. The DOD "Hazard Studies" are based on latest available intelligence estimates. They integrate and analyze factors of probable enemy deployment of weapons, weapon and delivery characteristics, U. S. active defense effectiveness, climatological factors, etc. The results are presented in the form of "Blast and Fallout Probability" charts, one of which is illustrated in Fig. 2-1. On that chart, for example, it is shown that there is 62 percent probability that blast will be less than 120 psi and 74 percent that nuclear radiation will be less than 10,000 r/hr (theoretical H + 1 hr. rate). The probability that neither will be exceeded is 50 percent (summation of all increments in the quadrant bounded by the limiting overpressure and radiation). Charts for nearly all CONUS military facilities, by Service, are part of the document, "Nuclear Attack Hazard on CONUS", prepared by OASD, Installations and Logistics. These charts will be distributed separately to recipients of this guide. The general pattern of attack on which these charts are based is also part of the I & L document. Because the charts reflect certain fixed assumptions of active defense capability, there will be anomalies in the individual cases. To determine the levels of protection required to attain any desired survival probability, the chart may be used directly with linear interpolation between levels of protection. For example, if a 70 percent survival is desired against both blast and fallout, the levels of protection required would be approximately 240 psi overpressure and 10,000 r/hr, at H + 1.

2.3.3 Further Analysis. It is apparent from the above illustration that the desired survival probability is not always practically attainable. Then it would be necessary to determine what survival can reasonably be attained and whether this is still adequate. A plot of "survival" vs. "level of protection" can be made quickly from the "Blast and Fallout" charts of Fig. 2-1. A typical plot of this type is shown in Fig. 2-2.

It is readily apparent that the greatest benefit accrues in the range 50-100 psi. At 50 psi, however, the survival is probably too low (30%). Thus a reasonable approximation is that 80 psi protection would provide at least 50 percent survival and would probably be optimum in this illustrative case. This can be refined somewhat in terms of cost, by using the curves of Fig. 2-3, and even further if desired by analytical methods described in SECTION 5. Other basic alternatives in such cases are dispersal or duplication. Evaluation of the vulnerability of a remote or dispersed facility can be accomplished by the methods described in Para. 2.4

2.4 PROCEDURE B, GENERAL COMPUTATIONS

2.4.1 Establish the Desired Survival Probability. With some qualification the same method should be used in determining the Desired Survival Probability as was explained in Para. 2.3.1 for Procedure A. The principal difference between Procedures A and B in this regard is that Procedure A does not have to consider single versus multiple attacks since this element is inherent in the DOD Hazard Studies. Procedure B should take into account the number of attacks expected and the desired survival probabilities should be correlated with this number. The problem of achieving high survival probabilities against multiple attacks becomes formidable. It is therefore important that the estimated number of attacks should be reasonable.

While it is not generally possible to set down rules for estimating the number of attacks, it is sometimes possible to determine from the function of the facility the number of attacks which must be withstood. Some installations which might have only one-time use, in retaliation for example, need only survive the initial attack. Other installations might need to survive repeated attacks.

2.4.2 Estimating the Attack

(a) Discussion. It is necessary to have the best estimate possible as to what the intentions and capabilities of a potential enemy are in order to provide protection. To do this intelligently, one should know the general strategy to be used by an enemy, his purpose in attacking, his knowledge of our installations and capabilities, and many other somewhat intangible factors. There are in addition the technical features of an enemy's capabilities which must be considered: the numbers and size of his weapon stockpile; the methods of delivery available to him; the accuracy of his delivery; the reliability of his weapons.

A planner in a rather localized situation cannot hope to have complete and current information of this type. However, estimates may be available through the service intelligence agencies. In case of controversy, the National Intelligence Estimates, available through JCS, will control.

(b) Factors. The following factors must be considered.

(1) Weapon Size (W) and Number of Attacks (n). An enemy's weapons should be assumed to be at least equal to our own. The yields should be reasonable for the time during which the structure has its useful life. Yields of 1 MT, 10 MT, and 50 MT are believed to be realistic for perhaps the next ten years. As for the number of weapons, it should be assumed that weapons will be delivered in the most efficient manner, i.e., not simultaneously at a single aiming point, and with multiple attacks dispersed to avoid "over killing."

(2) Method of Delivery. It may be assumed that the 1 MT and the 10 MT weapons can be delivered either by manned aircraft or guided missiles, while the 50 MT weapon is more likely to be delivered by aircraft.

(3) CEP, Probable Circular Error. It is assumed that errors are equally probable in all directions from the DGZ and that the errors will follow a standard Gaussian distribution. Values of CEP are uncertain and unpredictable. Current estimates vary between limits of the order of 0.5 to 3.0 nautical miles.

(4) Designated Ground Zero. The DGZ is taken to be at the site of a single target if isolated, a vital component in an installation, or the point which would maximize damage.

(5) Probability of Success of Attack (Z). Some estimate should be made of the probability of any given attack launched by an enemy reaching a target area. This would take into account the possibility of mechanical failures, aborts, and active defenses. These factors are difficult to assess, but if no better information is available, Z may be taken as 75 percent for aircraft and 50 percent for missiles. In the calculations described for Procedure B the value of Z is taken as unity. For other values of Z the survival probability obtained for Z = 1.00 can be modified:

$$S_Z = 1 - Z(1 - S_{Z=1})$$

(c) Uncertainties Described by Probabilities. In this report the uncertain elements described by probabilities are all related to the weapon, its delivery, and the pressures which will prevail. No element of probability is associated with the ability of a structure to withstand its limit design load, although obviously some probability does exist.

In the case where the planner makes his own Estimate of Attack, the uncertainties are in his assumptions of W, CEP, n, and probable location of the DGZ. The probability of survival of a structure is then the probability that the limit-design loads of the structure will not be exceeded. Another way of saying this is that it is the probability that GZ will not be too close to the structure.

2.4.3 Basic Single-Shot Probabilities. Having a set of weapon parameters, e.g. weapon yield (W), CEP, aiming point (DGZ), probability of

arrival (Z), and a set of target parameters for the facility, e.g. hardness, R_v, and the relation of target to aiming point, it is possible to compute the probability of survival of the facility from a one-weapon attack. In some analyses this single-shot survival probability may be the answer desired. In many cases the single-shot probabilities can be compounded to study complex installations and multiple-weapon attacks.

The fundamental assumption which will be used in determining single-shot probabilities is that the errors of weapon delivery will be distributed symmetrically about the aiming point. This assumption is reasonable for missiles having an extremely lofty trajectory and it is fairly accurate for weapons delivered by high-flying manned bombers, especially if the direction of flight of the aircraft is unpredictable. Thus if a large number of weapons were directed at the same aiming point, they would fall in a circular distribution pattern around the aiming point. The greatest density of hits would lie closest to the aiming point. An idealized pattern of this type can be described mathematically by the Gaussian Circular Distribution Function. The CEP is then the radius of the circle about the aiming point within which one half of the hits will occur. The entire Gaussian pattern, i.e., the area within which almost 100 percent of the hits will fall, has a radius of slightly over three CEP's.

Any given area in the Gaussian pattern can be evaluated to show the percentage of total hits which will normally fall within the area. From the target with its known hardness and the expected weapon size (yield), one can determine the radius of vulnerability about the target. This radius forms a circle, within which a hit by the assumed weapon will cause over-pressures at the target in excess of its hardness. All of these elements can be combined to give a calculated probability of a "hit", or conversely the probability of survival.

Three methods of obtaining the single-shot probabilities will be discussed.

(a) <u>Analytical</u>. For a target which is at DGZ, the vulnerability radius, survival probability of the single target from a single weapon, and CEP are related by the equation:

$$ {}_1S_1 = 0.5^{(R_v/\mathrm{CEP})^2} \tag{2-1} $$

This equation is plotted in Fig. 2-4 and 2-5 for selected values of R_v and CEP.

For a target which is a distance L from DGZ, the analytical solution is usually difficult. An approximate analytical expression is:

$$ {}_1S_1 = 1 - \left[\frac{2.2(R_v/\mathrm{CEP})}{(L/\mathrm{CEP})^3 + 3}\right]^2 \tag{2-2} $$

For finding ${}_1S_1$ Eq. (2-2) will usually be less accurate than the graphical solution.

(b) Graphical. The graphical solution has the merits of simplicity, flexibility, ease of understanding, and sufficient accuracy. It requires estimation of CEP, DGZ, and L as do the other methods. It has the disadvantage of requiring trial-and-error solution to find the value of R_v corresponding to a desired survival probability.

Figure 2-6 is a plot of cells of equal probability which is a graphical representation of the Gaussian Circular Probability Function. The center of the plot is the DGZ. The "squares" vary in size, the smallest being near the center. Each square has a value which represents the probability of GZ falling in the square. The plot is scaled in terms of CEP, half the squares lying within a circle of radius CEP. The entire plot covers an area slightly larger than three CEP's in radius.

It is only necessary to locate the target in the plot in relation to the DGZ, all distances being expressed as multiples of the CEP. Around the target the vulnerability circle is drawn, also to the scale of the CEP. A value N is obtained by counting appropriate squares and multiplying by the values of the squares, estimating and counting fractions of squares also. N represents the probability of a hit in the vulnerability circle and ${}_1S_1 = 1 - N$.

(c) Air Force Nomogram. The Air Force Physical Vulnerability Division has prepared for use in this volume a nomographic chart which will give the probability of a certain effect at a given distance from DGZ. This chart is given herein as Fig. 2-15. It should be noted that the chart is based upon a Sigma of 10 percent. Sigma is the ratio of the standard deviation of the damage-probability curve to the vulnerability radius. This is in contrast to the other procedures and methods described herein which are based upon a sigma of zero, or a "cookie-cutter" concept. The difference in the results for the two values of sigma is usually not significant and certainly introduces no unduly large error in the type of problem treated here.

2.4.4 Determining Level of Protection Against a Single-Weapon Attack. The probabilities found in Para. 2.4.3 can be applied directly to several situations. The problem will generally be one of two types: calculating the survival probability from the other values which are known or assumed; or, calculating the overpressure at the structural site (and thus the required hardening) from the other values which are known or assumed. In this Review Guide the emphasis is on the latter problem, i.e., determining the hardness required.

Case 1--Single facility which is to be built at DGZ.

In this case the facility is the aiming point and would be plotted at the center of the Gaussian Distribution Chart (Fig. 2-6).

The known or assumed starting values are W, CEP, S', and L (=0). Either Eq. (2-1) or the graphical solution of Para. 2.4.3(b) can be used to compute R_v. Figure 2-7 can then be entered with W and R_v to find the overpressure p_{so} for which the structure must be hardened.

An alternate solution would be to use the chart in Fig. 2-15.

Case 2--Single facility which is to be built near the DGZ.

In this case the facility is not itself the aiming point, but is sufficiently close to the DGZ so as to have a probability of being hit.

The known or assumed starting values are W, CEP, S', and L. Equation (2-2) or the graphical solution of Para. 2.4.3(b) can be used to compute R_v. Equation (2-2) is less accurate than the graphical solution but gives a reasonable answer if L/CEP is less than 1.

The graphical method is more accurate but has the disadvantage of being a trial-and-error procedure, the problem being to find the radius of a circle whose center is a distance L from DGZ and whose area corresponds to the desired survival probability. The radius of this circle is the R_v sought.

Having found the value of R_v, Fig. 2-7 can be used to find the overpressure in the same manner as was done in Case 1.

The chart in Fig. 2-15 can be used as an alternate and is well well suited for this case.

Case 3--Two or more dependent, overlapping targets subjected to a single weapon attack.

If two or more targets have overlapping vulnerability circles, it is possible that one weapon could fall in a vulnerability area common to two or more and thus destroy that number. It is also possible that the single weapon could fall in a vulnerability area exclusively related to one target and would thus destroy only that one target.

This case is well-suited to graphical solution. The targets with their vulnerability circles drawn to the scale of the CEP can be laid out geometrically on an overlay. This overlay can be placed on the Gaussian Distribution Chart (Fig. 2-6) in different positions corresponding to different DGZ's, and the optimum DGZ from the enemy viewpoint selected (if not otherwise known). The probabilities of hits in the various vulnerability areas can then be determined (N_A, N_B, N_{AB}). For example, if there are two targets, A and B:

Probability of Survival of A is:

$${}_1S_A = 1 - N_A$$

Probability of survival of B is:

$${}_1S_B = 1 - N_B$$

Probability that neither will survive one weapon is:

$${}_1S_0 = 1 - N_{AB}$$

where N_{AB} is the N for the overlapping area

Probability that both will survive one weapon is:

$${}_1S_{AB} = 1 - (N_A + N_B - N_{AB}) .$$

<u>Case 4--Two or more non-overlapping targets subjected to a single weapon attack</u>.

These targets can be either independent or dependent. The problem for each target is that of Case 1 or Case 2.

Any two targets are independent if their separation distance, d, is equal to or greater than 6.4 CEP plus the sum of their vulnerability radii. This means that for practical purposes only one can be plotted at a time on the Gaussian Distribution Chart.

Any two targets are dependent when the condition stated in the preceding paragraph is not satisfied. Although no more than one can be hit by a given weapon, all have a chance of being hit, and all would show on the Gaussian Distribution Plot. For a given DGZ the probability of hitting at least one of them (unspecified) with one weapon would be the sum of the individual probabilities of hitting each one of them with one weapon.

2.4.5 <u>Multiple-Shot Probabilities</u>. If more than one weapon is delivered against the target (or targets) the probabilities change, and the survival probabilities will be less than those computed for single weapon attack. The multiple-shot probabilities can be computed from the basic single-shot probabilities.

The parameters involved are the number of targets m, the number of weapons n, the weapon yield W, the DGZ, the CEP, the number of hits h, and the configuration of the target(s) and DGZ. All of the parameters can vary but for the purpose of this discussion it is assumed that the [illegible] and the yield W remain the same for each of the n attacks.

The problem involved here is best solved in reverse. That is, although the level of hardness required for a certain survival probability is the desired answer, it is easier to assume levels of hardness and solve for the resulting survival probabilities. It may be desirable to plot the results for different values of parameters, and then use the curves to arrive at the hardness required.

The graphical solution is best suited for this situation. The target will generally be a complex consisting of several vulnerable points. It will probably involve duplicated portions, moderate dispersion, and varying degrees of hardness. One must carefully define what is necessary for survival. For example, in an underground complex with two identical entrances, the survival of either tunnel along with the facility itself might be considered to be an over-all survival.

Since there are so many parameters involved in the multiple-target, multiple-shot problem, the chief benefit may be in making qualitative comparisons of different situations rather than attempting to assign a precise number to any given situation. The next paragraph discusses determination of probabilities for several different cases, and illustrative examples are given in Para. 2.8.

2.4.6 Determining Level of Protection Against a Multiple-Weapon Attack. Four cases will be considered.

Case 5--Single target subjected to multiple-weapon attacks.

For n attacks the survival probability becomes:

$$ {}_nS_1 = {}_1S_1^n $$

Knowing n and the desired ${}_nS_1$, this equation can be solved for ${}_1S_1$. This value can then be used as in Case 1 or 2 as appropriate to find R_v. From R_v and CEP the value of overpressure can be obtained from Fig. 2-7 as before.

Case 6--Two or more overlapping targets subjected to multiple-weapon attacks.

These targets are also dependent since any given weapon has a finite probability of hitting any one of the targets.

This procedure cannot be standardized but the solution is as follows. The facility is plotted in units of the CEP on an overlay if the DGZ is uncertain or directly on the Gaussian Distribution Chart if the DGZ is established. The single-shot hit probabilities, N, are determined for all segments formed by the circles of vulnerability.

The survival criterion must be set, i.e., precisely which ones and how many of the targets must survive in order to be considered an over-all survival for the complex. From this the converse occurrence, i.e., an enemy success, can be defined. It is then necessary to look at all possible mutually exclusive ways of enemy success with n weapons. The total probability of enemy success is the sum of the probabilities of the mutually exclusive ways. The survival probability may be found by subtracting the total enemy success probability from unity.

Case 7-- Two or more dependent, non-overlapping targets subjected to multiple-weapon attacks.

In this case each target has a finite probability of being hit by any given weapon but no weapon can hit more than one target.

The survival and enemy success criteria should be defined as was done in Case 6. The individual single-shot hit probabilities, N, should be determined by plotting the configuration on the Gaussian Distribution Chart. The probabilities can then be found. For example, if there are three targets A, B, and C for which the single-shot hit probabilities are N_A, N_B, and N_C, the following are typical hit probabilities which can be formed:

n	Prob. of only A being hit	Prob. of A and B being hit but not C	Prob. of all three being hit
1	N_A	0	0
2	$2!(N_A)[1-N_B-N_C]$	$2!(N_A)(N_B)$	0
3	$\frac{3!}{2!}(N_A)[1-N_b-N_c]^2$	$3!(N_A)(N_B)(1-N_C)$	$3!(N_A)(N_B)(N_C)$
4	$\frac{4!}{3!}(N_A)[1-N_B-N_C]^3$	$\frac{4!}{2!}(N_A)(N_B)(1-N_C)^2$	$4!(N_A)(N_B)(N_C)$

If the targets have equal probabilities of being hit, i.e., $N_A = N_B = N_C$, the above relationships become more simple.

Case 8--Two or more independent non-overlapping targets subjected to multiple-weapon attacks.

In this case each target has a finite chance of being hit only by the weapon directed at it (or at a nearby DGZ). Other targets have a zero probability of being hit by this weapon. The quantity n is therefore used to designate the number of weapons which have a possibility of hitting a given target, and it is further assumed that n is the same for all targets. In other words, if there are m targets and n weapons are launched at each, the total number of weapons is mn.

If m targets each have a survival probability against a single attack ${}_1S_1$, and if n weapons are launched against each of them, the following relations hold:

Survival Probabilities	Number of Targets			
	$m = 1$	$m = 2$	$m = 3$	$m = 4$
Prob. of none surviving n attacks (${}_nS_0$)	$(1-{}_1S_1^n)$	$(1-{}_1S_1^n)^2$	$(1-{}_1S_1^n)^3$	$(1-{}_1S_1^n)^4$
Prob. of one surviving n attacks (${}_nS_1$)	${}_1S_1^n$	$2\,{}_1S_1^n(1-{}_1S_1^n)$	$3\,{}_1S_1^n(1-{}_1S_1^n)^2$	$4\,{}_1S_1^n(1-{}_1S_1^n)^3$
Prob. of two surviving n attacks (${}_nS_2$)		${}_1S_1^{2n}$	$3\,{}_1S_1^{2n}(1-{}_1S_1^n)$	$6\,{}_1S_1^{2n}(1-{}_1S_1^n)^2$
Prob. of three surviving n attacks (${}_nS_3$)			${}_1S_1^{3n}$	$4\,{}_1S_1^{3n}(1-{}_1S_1^n)$
Prob. of four surviving n attacks (${}_nS_4$)				${}_1S_1^{4n}$

It is seen that the probability expressions in any column under a value of m are terms in the expansion of the binominal expression,

$$[(1 - {}_1S_1^n) + {}_1S_1^n]^m$$

and also that the sum of the terms in this expansion total unity. Further the summations of terms starting with the last, then the last plus the next to last, etc., give the probabilities of m surviving, at least (m - 1) surviving, etc. This reverse summation can be expressed mathematically as:

$${}_nS_{\geq r} = {}_1S_1^{mn}\left[1 + \sum_{a=1}^{(m-r)} \frac{m(m-1)\ldots(m-a+1)}{a!}\left\{\frac{1 - {}_1S_1^n}{{}_1S_1^n}\right\}^a\right]$$

where ${}_nS_{\geq r}$ is the probability that at least r out of m targets, each with single-shot survival probability of ${}_1S_1$, will survive n attacks. a is a summation index. n is the same for each target and can be any integer, one or greater.

2.5 EFFECTS OTHER THAN BLAST

2.5.1 Initial Nuclear Radiation. During the first few tens of seconds following a nuclear burst, large intensities of gamma and neutron radiation are produced. Various components of weapons systems and equipment, as well as personnel, are vulnerable to these effects. It may become important in target analysis to determine the expected level or dosage at a particular location.

Figure 2-8 is a plot of weapon yield versus range for various levels of total initial gamma dosage. ~~These curves were obtained from formulas con-~~ tained in Ref. 17. It should be noted that the gamma dosage does not scale simply with yield since the fission-produced gamma dosage from megaton yield shots is affected by hydrodynamic motions and cloud rise.

Figure 2-9 is a plot of weapon yield versus range for various levels of neutron flux. These curves can be computed from Ref. 17, also. Figures 2-8 and 2-9 are used in a manner analogous to Fig. 2-7 (Weapon Yield vs. Range vs. Overpressure) in determining a vulnerability radius, R_v. (For radiation the vulnerability radius will also be referred to as the "Lethal Range.") Once the value of R_v has been found, the computation of probabilities can be made as was done for overpressures. These calculations can be extended to consider multiple-weapon attacks as was done with the case of overpressure probabilities.

2.5.2 Fallout. All of the foregoing analyses pertain to effects of blast and related initial radiation. In order to determine the fallout intensity, reference must be made to ENW or CAW, (Refs. 1 and 2) which permit determination of intensity and total dose for a set of assumed geographic and wind conditions. To determine the probability of occurrence of these conditions it is necessary to consider the incidence of winds at different times of the year. For purposes of this document, analysis will be made on the basis of a burst in an upwind direction from the facility at a distance consistent with the design overpressure and weapon yield. This approach is a departure in principle from that discussed previously for blast and prompt radiation since it considers the worst case rather than specific probabilities. However, in general, protection against fallout will not be critical in design.

The cumulative fallout in residual gamma radiation can be determined for locations downwind from DGZ by use of Figs. 2-10 and 2-11. The downwind situation is taken since it is the most severe condition for design. Figure 2-10 gives values of One-Hour Reference Dose Rates for various weapon yields and distances for a 15-knot scaling wind. The values of Dosage Rates can be multiplied by factors shown on Fig. 2-10 for other wind velocities.

The time of arrival at the fallout site is obtained by dividing the distance by the wind velocity. The cumulative fallout occurring at the site between the arrival time and any desired time is obtained from Fig. 2-11. Enter with both values of time and read from the ordinate scale the two values of the ratio of Total Dose R to the One-Hour Reference Dose Rate. The difference between these two ratio values multiplied by the One-Hour Reference

Dose Rate, previously obtained from Fig. 2-10 gives the total fallout dose in the interval between the two time entries.

2.5.3 Ground Shock. In certain situations involving below-ground facilities the critical effect of nuclear attack may be the ground shock imparted to the structure or its contents. As explained in SECTION 5 this ground shock is a function of geometry, geology, and weapon parameters. The characteristic accelerations, velocities, and displacements in the earth at the point of interest can be predicted by the methods of SECTION 5. The occurrence of a specific kind and magnitude of ground motion can therefore be correlated with the expected weapon and with a peak side-on overpressure at some point at the ground surface. The probability of the ground shock occurrence can then be taken to be the same as the probability that the related overpressure will occur at the point on the ground surface. The latter probability can be found by the methods of this section.

2.5.4 Thermal Considerations. The effects of thermal radiation are normally not the governing criteria in structural target analysis. However, it is important to know the thermal levels at various distances from a weapon detonation since any aboveground exposed ventilators, doors, etc., might be vulnerable. Figure 2-12 shows the thermal radiation received at various distances per unit area exposed at normal incidence for several weapon yields. Since this is a function of the atmospheric conditions, the data shown are selected for a visibility of 50 miles. This is an optimum situation for thermal radiation. Vulnerabilities are discussed in SECTION 4.

2.5.5 Biological and Chemical Agents. It is assumed that any facility which is a target or within blast effect distance of a target may also be subjected to biological and chemical agents. Appropriate protection by filtration and sealing of the structure is required.

2.6 DISPERSAL

2.6.1 Determination of Siting. The distance from DGZ required in order to reduce blast or fallout hazard is determined by the methods described in Para. 2.4.

2.6.2 General Considerations. When new sites are being considered the following conditions should be effected if possible:

If the new installation itself is a recognized target, it must be located away from centers of population, if it is possible to do so. A rule of thumb which may be used is to so locate the new installation that a circle of 60-mile radius, centered on a point 40 miles in the known prevailing downwind direction from the site (or due East if this direction

not known), will not contain centers of population of more than 50,000 people. More refined analyses may be used based on projections of probable fallout patterns, should this rule of thumb be impracticable to apply.

2.7 SUMMATION

After completion of the target analysis one of the three following conditions will prevail:

a. A firm level of protection will have been established to meet the desired survival requirement.

b. A compromise level of protection will have been adopted with appropriate curves showing relationship of level of protection to survival. These will be carried on to the next step in the review process.

c. It will have been concluded that "hardening" is not feasible as a means of protection.

2.8 ILLUSTRATIVE EXAMPLES

The following illustrations show the application of Procedure B to the eight cases presented in Para. 2.4.4 and Para. 2.4.6.

2.8.1 Example 1 (Case 1)

A single facility is to be built. It is considered important enough to be a target itself and is thus assumed to be at the DGZ. The expected attack is by one 10 MT weapon delivered with a CEP of 20,000 ft. The desired single-shot survival probability is 90%. Find the hardness required for the structure.

(a) Analytical Solution.

From Eq. (2-1)

$$ {}_1S_1' = 0.5^{(R_v/CEP)^2} $$

$$ 0.90 = 0.5^{(R_v/20,000)^2} $$

$$ R_v = 7,800 \text{ ft.} $$

From Fig. ~~2-6~~ 2-7

For R_v = 7.8 thousand ft. and W = 10 MT

P_{so} = 95 psi, say 100 psi

(b) Graphical Solution. A hit probability of 0.10 is obtained by subtracting $_1S_1$ from unity. From Fig. 2-6 it is seen that $N = 0.10$ for a circle having a radius R_v slightly greater than the second ring. This distance is approximately 0.4 CEP; therefore R_v = (0.4)(20,000 ft.) = 8,000 ft. Entering Fig. 2-7 with W = 10 MT and R_v = 8,000 ft., a value of 90 psi is obtained.

(c) Solution using Fig. 2-15. Since the desired survival probability is 90%, there must be a 90% probability that the overpressure will not exceed the hardness of the structure. Conversely, there must be a 10% probability that the overpressure will be equal to or greater than the hardness of the structure. The entering values for Fig. 2-15 are: CEP = 20,000 ft., distance from DGZ = 0, W = 10 MT, Prob. = 0.10. It is seen that this problem does not fall within the range of values of Fig. 2-15 and therefore cannot be solved by this method.

2.8.2 Example 2 (Case 2). A single facility is to be built 10,000 ft. from a known DGZ. The facility itself is not considered sufficiently important to cause an enemy to change the DGZ. The expected attack is by one 8 MT weapon delivered with a CEP of 6000 ft. The desired survival probability is 80%. Find the required degree of hardness for the facility.

(a) Analytical Solution. This problem could be solved by using Eq. (2-2). However, the ratio of L/CEP here is 1.67 and Eq. (2-2) becomes inaccurate for L/CEP > 1. Therefore, the analytical solution is not preferred.

(b) Graphical Solution. Solution by plotting in Fig. 2-6 would be possible by trial and error. It is necessary to find the radius R_v required for a circle which: 1) has its center a distance L/CEP from the center of the plot, and 2) has an area which includes "squares" having a value of 0.20. Having found R_v, Fig. 2-7 can be entered to find the level of hardening. For this situation the Fig. 2-15 solution is simpler and more direct.

(c) Solution using Fig. 2-15 (Surface). Entering the nomograph with the proper values of W, CEP, and L, the value of p_{so} = 100 psi can be read.

2.8.3 Example 3 (Case 3). Two facilities, A and B, are 10,000 ft. apart and Facility A is at the DGZ. The expected attack is by one 12 MT weapon delivered with a CEP of 20,000 ft. If Facility A is hardened to 85 psi and Facility B is hardened to 20 psi, what are the probabilities of each to survive, of both to survive, and of neither to survive.

From Fig. 2-7

For A R_v = 8,000 ft.

R_v/CEP = 0.4

For B R_v = 16,000 ft.

R_v/CEP = 0.8

Plotting on the Gaussian Distribution Chart (See Fig. 2-13), the following numbers of "squares" are counted:

N_A = 0.100 (one hundred "squares" of value 0.001)

N_B = 0.300 (three hundred "squares" of value 0.001)

N_{AB} = 0.087 (87 "squares" of value 0.001)

Probability that A will survive

$${}_1S_A = 1 - 0.100 = 0.900 \quad \text{or } \underline{90\%}$$

Probability that B will survive

$${}_1S_B = 1 - 0.300 = 0.700 \quad \text{or } \underline{70\%}$$

Probability that both will survive

$${}_1S_{A+B} = 1 - (0.100 + 0.300 - 0.087) = 0.687 \text{ or } \underline{69\%}$$

Probability that neither will survive

$${}_1S_0 = 0.087 \quad \text{or } \underline{8.7\%}$$

2.8.4 Example 4 (Case 4). Three facilities, A, B, and C are located with respect to the DGZ so that for a single weapon attack assumed Facility A has a survival probability of 90%, Facility B has a survival probability of 65%, and Facility C has a survival probability of 75%. What are the probabilities of either A or B being hit, either B or C, any one of the three?

The hit probabilities are

$$H_A = 1 - 0.90 = 0.10$$
$$H_B = 1 - 0.65 = 0.35$$
$$H_C = 1 - 0.75 = 0.25$$

Probability of either A or B being hit

$$H_{A \text{ or } B} = 0.10 + 0.35 = 0.45 \quad \text{or } \underline{45\%}$$

Probability of either B or C being hit

$$H_{B \text{ or } C} = 0.35 + 0.25 = 0.60 \quad \text{or } \underline{60\%}$$

Probability of any one being hit

$$H_{A,B \text{ or } C} = 0.10 + 0.35 + 0.25 = 0.70 \quad \text{or } \underline{70\%}$$

2.8.5 Example 5 (Case 5). If a facility is located 10,000 ft. from the DGZ of an 8 MT weapon, and assuming a CEP of 20,000 ft., what design pressure level is required to achieve a survival probability of 90%?

Solving Eq. (2.2) for R_v

$$R_v = 0.45 \text{ CEP} \left[\left(\frac{L}{CEP}\right)^3 + 3 \right] (1 - {}_1S_1)^{1/2}$$

$$= 8,880 \text{ ft.}$$

From Fig. 2-7; $P_{so} \simeq$ 55 psi

What is the survival probability for this facility if a total of three identical attacks were made?

$${}_3S_1 = (0.90)^3 = 0.729 \text{ or } 73\%$$

What design pressure level is required to achieve a 90% survival probability from the three attacks?

$$0.90 = ({}_1S_1)^3$$

$${}_1S_1 = 0.966$$

Substituting 0.966 for ${}_1S_1$ in Eq. (2.2);

$$R_v = 5180 \text{ ft.}$$

From Fig. 2-7; $P_{so} \simeq$ 220 psi

2.8.6 Example 6 (Case 6). Three similar installations, A, B, and C are each hardened to 12 psi and located so that they form the vertices of an equilateral triangle 3000 ft. on a side. The estimated attack is from one or more 100 KT weapons each with a CEP of 10,000 ft. The DGZ is not known but it is assumed that all weapons delivered will use the same DGZ since the facilities are close together. Find: the probability that all will survive 4 attacks; the probability that both A and B will survive 4 attacks; the probability that A will survive 4 attacks; the probability that at least one will be a hit.

From Fig. 2-7 using 12 psi and 100 KT

$$R_v = 4150 \text{ ft. (for each target)}$$

$$R_v/CEP = \frac{4,150}{10,000} = 0.415$$

If the three installations are plotted to the scale of the CEP on a transparent overlay and the overlay placed on the Gaussian Distribution Chart (Fig. 2-6) it is found that the DGZ can be anywhere within the triangle formed by the three installations without changing significantly their survival probabilities. Therefore, for ease of computation the DGZ is taken at the geometrical center of the triangle (Fig. 2-14).

The following single-shot probabilities are obtained by counting "squares":

$$
\begin{aligned}
&\text{Only A, } N_A = 0.063 \\
&\text{Only B, } N_B = 0.063 \\
&\text{Only C, } N_C = 0.063 \\
&\text{A+B, } N_{A+B} = 0.016 \\
&\text{B+C, } N_{B+C} = 0.016 \\
&\text{A+C, } N_{A+C} = 0.016 \\
&\text{A+B+C, } N_{ABC} = 0.015 \\
&\text{A complete miss} = 1 - \left[(3)(0.063) - (3)(0.016) + 0.015\right] \\
&\qquad = 1 - 0.156 = 0.844
\end{aligned}
$$

<u>Probability that all will survive 4 attacks</u>

$$ {}_4S_3 = (0.844)^4 = 0.514 \quad \text{or } \underline{51\%} $$

<u>Probability that A and B will survive 4 attacks</u>

$$ {}_4S_{A+B} = \left\{ 1 - \left[(0.063)2 - (0.016)\right]\right\}^4 = 0.669 \quad \text{or } \underline{67\%} $$

<u>Probability that A will survive 4 attacks</u>

$$ {}_4S_A = \left\{1 - [0.063]\right\}^4 = 0.771 \quad \text{or } \underline{77\%} $$

<u>Probability that at least one will be hit</u>

$$ {}_4H_1 = [1 - 0.514] = 0.486 \quad \text{or } \underline{49\%} $$

2.8.7 <u>Example 7 (Case 7)</u>. Four identical installations each having a value of S of 80% are considered equally likely to be attacked. They are separated far enough to be non-overlapping, but they are still close enough together so that all have a probability of being hit when one of them is the DGZ. It is assumed that more than one attack will be launched simultaneously.

The single-shot probabilities have been determined to be as follows:

	N_A	N_B	N_C	N_D
A is DGZ	0.20	0.05	0.04	0.04
B is DGZ	0.05	0.20	0.10	0.01
C is DGZ	0.04	0.10	0.20	0.08
D is DGZ	0.04	0.01	0.08	0.20

Find the survival probabilities at the end of 5 attacks at a single DGZ.

Probability that none survive (all are hit)

If A is the DGZ	$5!(0.20)(0.05)(0.04)(0.04)(1)$	= Approx. zero
If B is the DGZ	$5!(0.05)(0.20)(0.10)(0.01)(1)$	= Approx. zero
If C is the DGZ	$5!(0.04)(0.10)(0.20)(0.08)(1)$	= Approx. zero
If D is the DGZ	$5!(0.04)(0.01)(0.08)(0.20)(1)$	= Approx. zero

Survival probability of A if four simultaneous attacks are launched, one aimed at each installation

$$S_A = [1 - (0.20 + 0.05 + 0.04 + 0.04)]$$

$$= 1 - 0.33 = 0.67 \quad \text{or } \underline{67\%}$$

Survival probability of A if two waves of four simultaneous attacks are launched (each installation has two weapons directed at it)

$${}_2S_A = (0.67)^2 = 0.446 \quad \text{or } \underline{45\%}$$

If A is the DGZ for three single attacks, what is the probability that A and B will be hit but not C (D may or may not be hit)

$${}_3H_{A+B} = 3!\ (N_A)(N_B)(1 - N_C)$$

$$= 6(0.20)(0.05)(0.96) = 0.0576 \quad \text{or } \underline{6\%}$$

If A is the DGZ for three single attacks, what is the probability that only A will be hit

$${}_3H_A = \frac{3!}{2}(N_A)(1 - N_A - N_B - N_C - N_D)^2$$

$$+ \frac{3!}{2}(N_A)^2(1 - N_A - N_B - N_C - N_D) + (N_A)^3$$

$$= 0.36 \quad \text{or } \underline{36\%}$$

2.8.8 Example 8 (Case 8). Four identical installations each having a value of ${}_1S_1^!$ of 80% are considered equally likely to be attacked. They are separated far enough to be considered independent targets, and are of course non-overlapping. It is probable that more than one attack will be launched either simultaneously or in sequence, but it is assumed that an equal number of weapons will be directed at each of the installations.

Find the survival probabilities at the end of five simultaneous attacks of each.

$m = 4$

$n = 5$

${}_1S_1 = 0.80$

Expand the expression $[(1 - 0.8^5) + 0.8^5]^4$

or $[0.622 + 0.378]^4$

$$[0.622 + 0.378]^4 = (0.622)^4 + 4(0.622)^3(0.378) + 6(0.622)^2(0.378)^2 + 4(0.622)(0.378)^3 + (0.378)^4$$

$$= \underset{\substack{\text{None}\\\text{Survive}}}{\underline{(0.150)}} + \underset{\substack{\text{Exactly}\\1\\\text{Survives}}}{\underline{(0.363)}} + \underset{\substack{\text{Exactly}\\2\\\text{Survive}}}{\underline{(0.333)}} + \underset{\substack{\text{Exactly}\\3\\\text{Survive}}}{\underline{(0.134)}} + \underset{\substack{\text{Exactly}\\4\\\text{Survive}}}{\underline{(0.020)}}$$

$$\begin{array}{ccccccccccc}
1.000 & = & 0.150 & + & 0.363 & + & 0.333 & + & 0.134 & + & 0.020 \\
 & & \underline{0.850} & = & 0.363 & + & 0.333 & + & 0.134 & + & 0.020 \\
 & & \text{at least 1 survives} & & \underline{0.487} & = & 0.333 & + & 0.134 & + & 0.020 \\
 & & & & \text{at least 2 survive} & & \underline{0.154} & + & 0.134 & + & 0.020 \\
 & & & & & & \text{at least 3 survive} & & \underline{0.020} & = & 0.020 \\
 & & & & & & & & \text{all 4 survive} & &
\end{array}$$

RADIATION INTENSITY r/hr. (H + 1 hr.)	PEAK OVERPRESSURES (PSI) Under 1.2	1.2-2.1	2.1-3.5	3.5-6	6-10	10-20	20-50	50-120	120-400	Over 400	RADIATION TOTALS	CUMULATIVE RADIATION TOTALS
Over 100,000	-	-	-	-	-	-	-	-	-	-	.00	1.00
30,000 - 100,000	-	-	-	-	-	-	-	-	-	-	.00	1.00
10,000 - 30,000	-	-	-	-	-	-	.06	.06	.06	.08	.26	1.00
3,000 - 10,000	-	-	.02	-	-	.02	.14	.28	.12	.12	.70	.74
1,000 - 3,000	-	-	-	-	-	.04	-	-	-	-	.04	.04
300 - 1,000	-	-	-	-	-	-	-	-	-	-	.00	.00
100 - 300	-	-	-	-	-	-	-	-	-	-	.00	.00
30 - 100	-	-	-	-	-	-	-	-	-	-	.00	.00
10 - 30	-	-	-	-	-	-	-	-	-	-	.00	.00
Under 10	-	-	-	-	-	-	-	-	-	-	.00	.00
OVERPRESSURE TOTALS	.00	.00	.02	.00	.00	.06	.20	.34	.18	.20	Note: Cumulative totals express probability that effect will be less than upper limit of appropriate row or column.	
CUMULATIVE OVERPRESSURE TOTALS	.00	.00	.02	.02	.02	.08	.28	.62	.80	1.00		

FIG. 2-1 BLAST AND FALLOUT PROBABILITIES FOR A SELECTED INSTALLATION

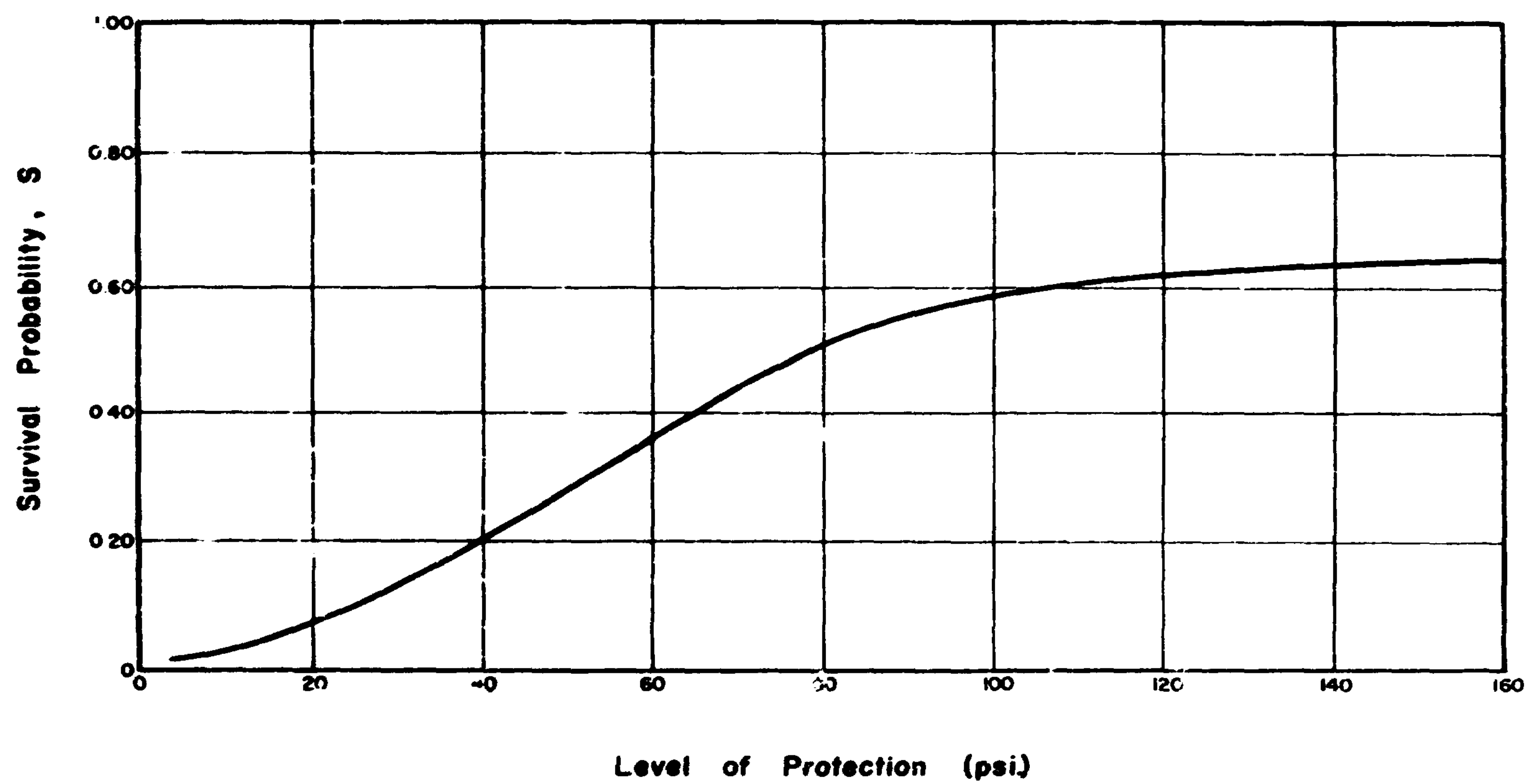


FIG. 2-2 SURVIVAL PROBABILITY VERSUS LEVEL OF PROTECTION FOR THE INSTALLATION DESCRIBED IN FIGURE 2-1

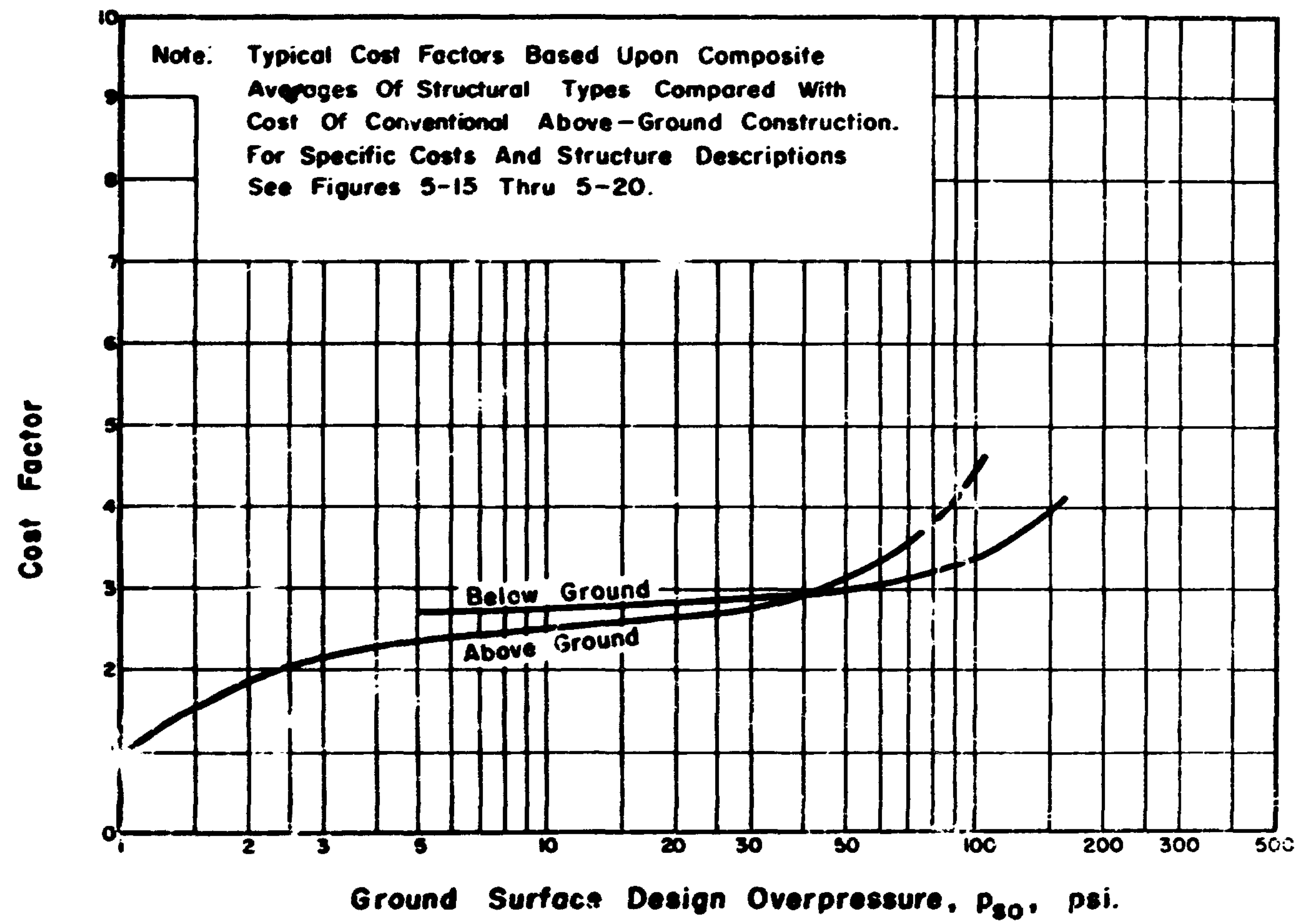


FIG. 2-3 COST FACTOR VERSUS DESIGN OVERPRESSURE

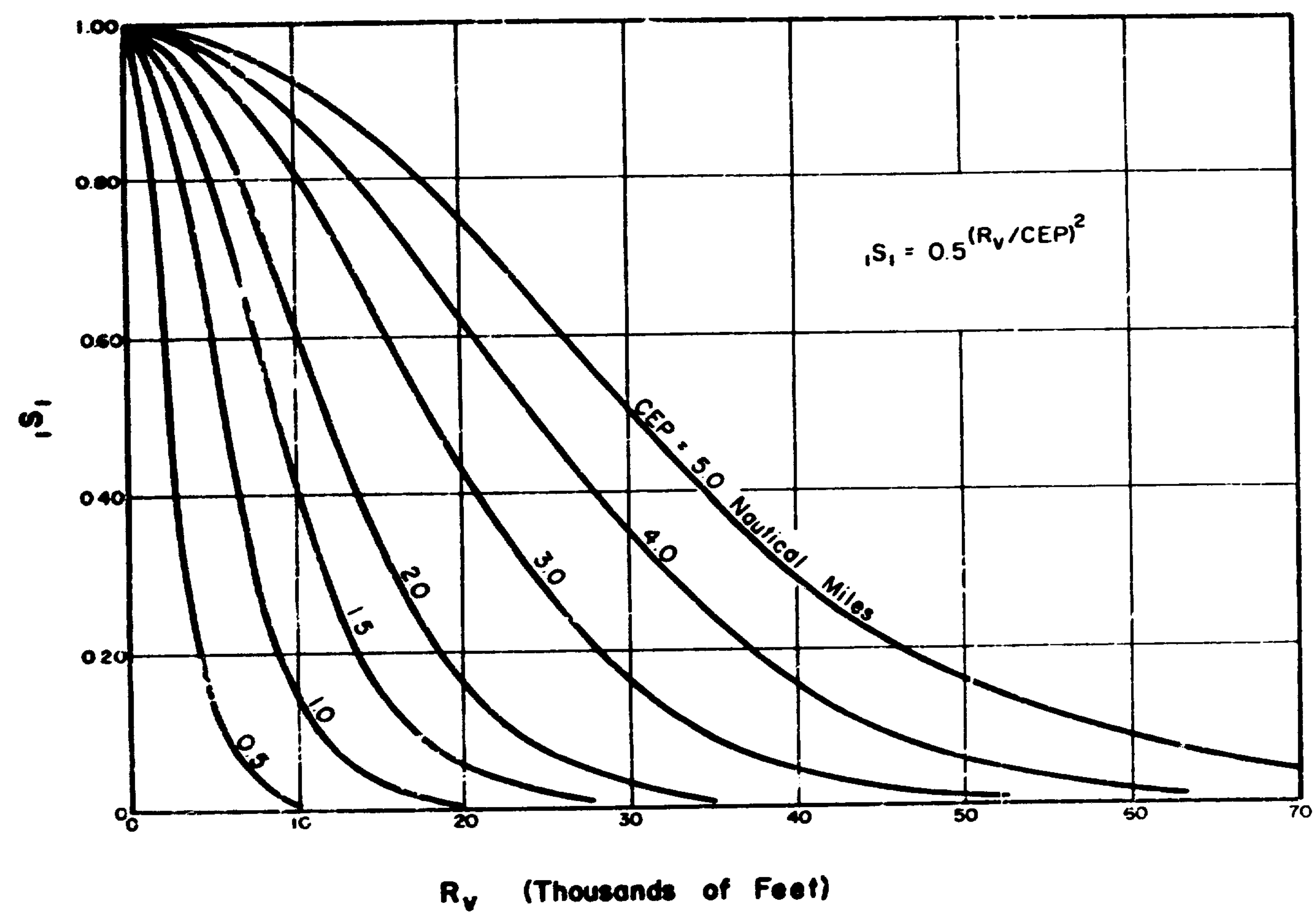


FIG. 2-4 SINGLE ATTACK SURVIVAL PROBABILITY VERSUS VULNERABILITY RADIUS FOR SINGLE TARGET WHICH IS AIMING POINT

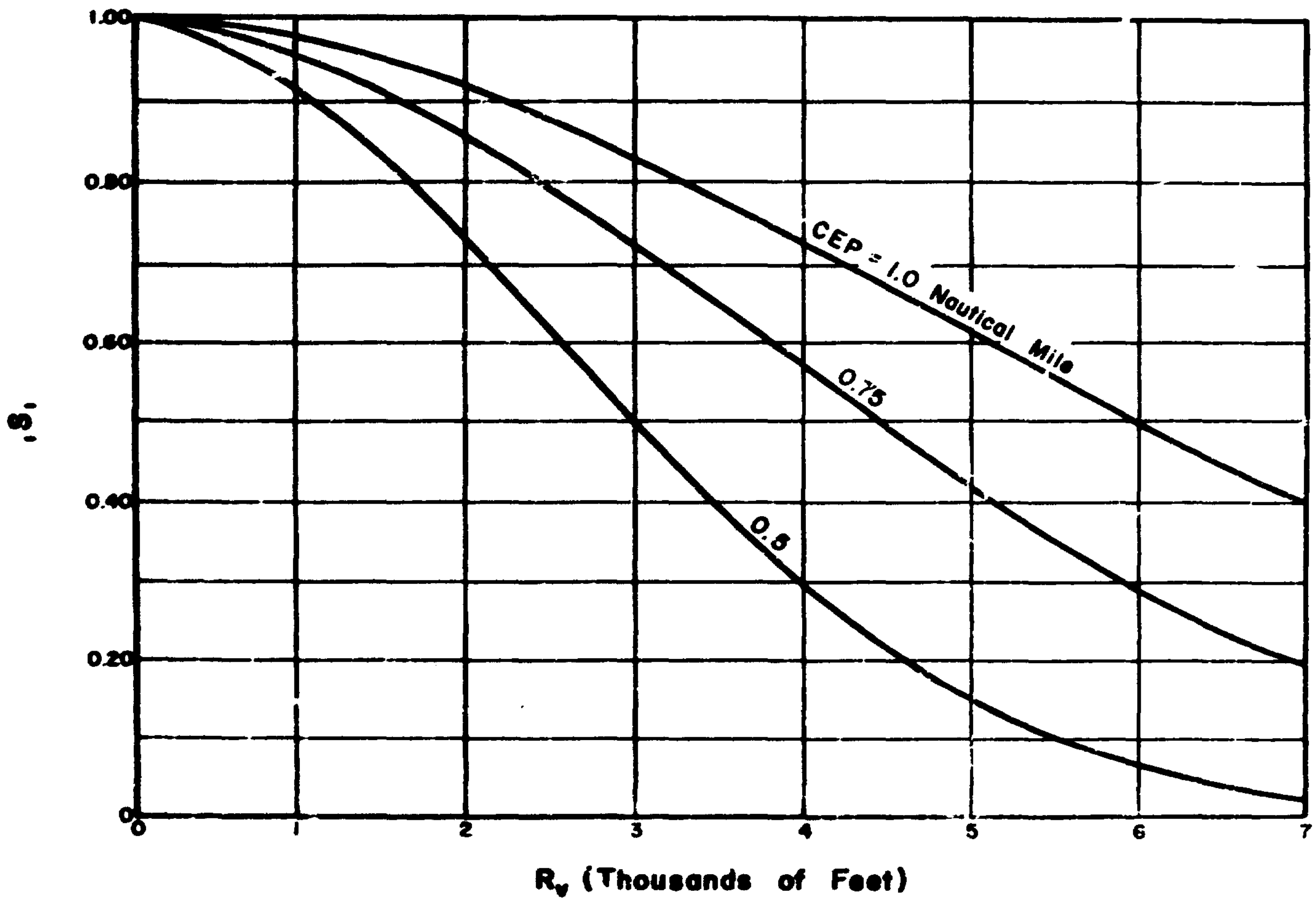
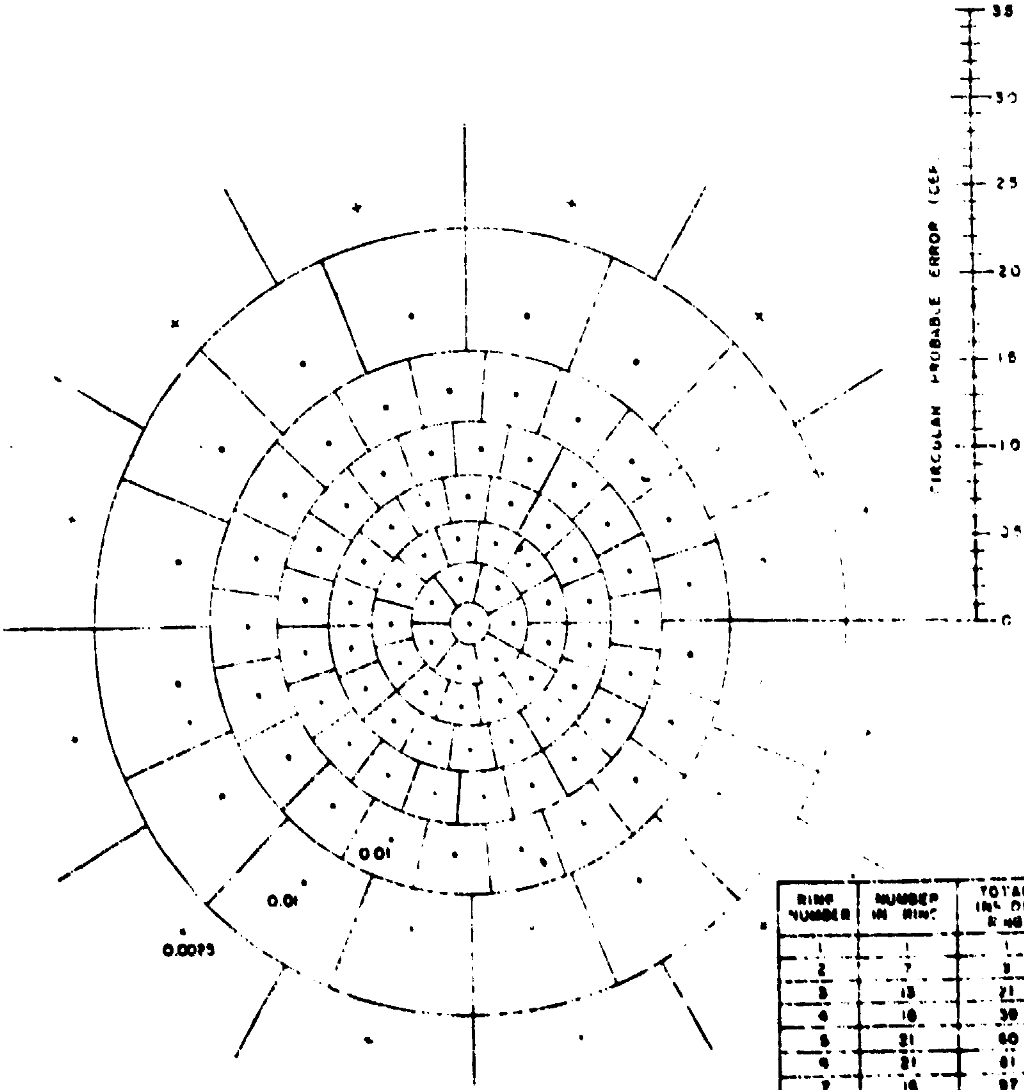


FIG. 2-5 SINGLE ATTACK SURVIVAL PROBABILITY VERSUS VULNERABILITY RADIUS FOR SINGLE TARGET WHICH IS AIMING POINT.



RING NUMBER	NUMBER IN RING	TOTAL INSIDE RING
1	1	1
2	7	8
3	13	21
4	18	39
5	21	60
6	21	81
7	16	97
8	18/4	100

FIG. 2-6 PLOT OF CELLS OF EQUAL PROBABILITY FOR CIRCULAR GAUSSIAN DISTRIBUTION

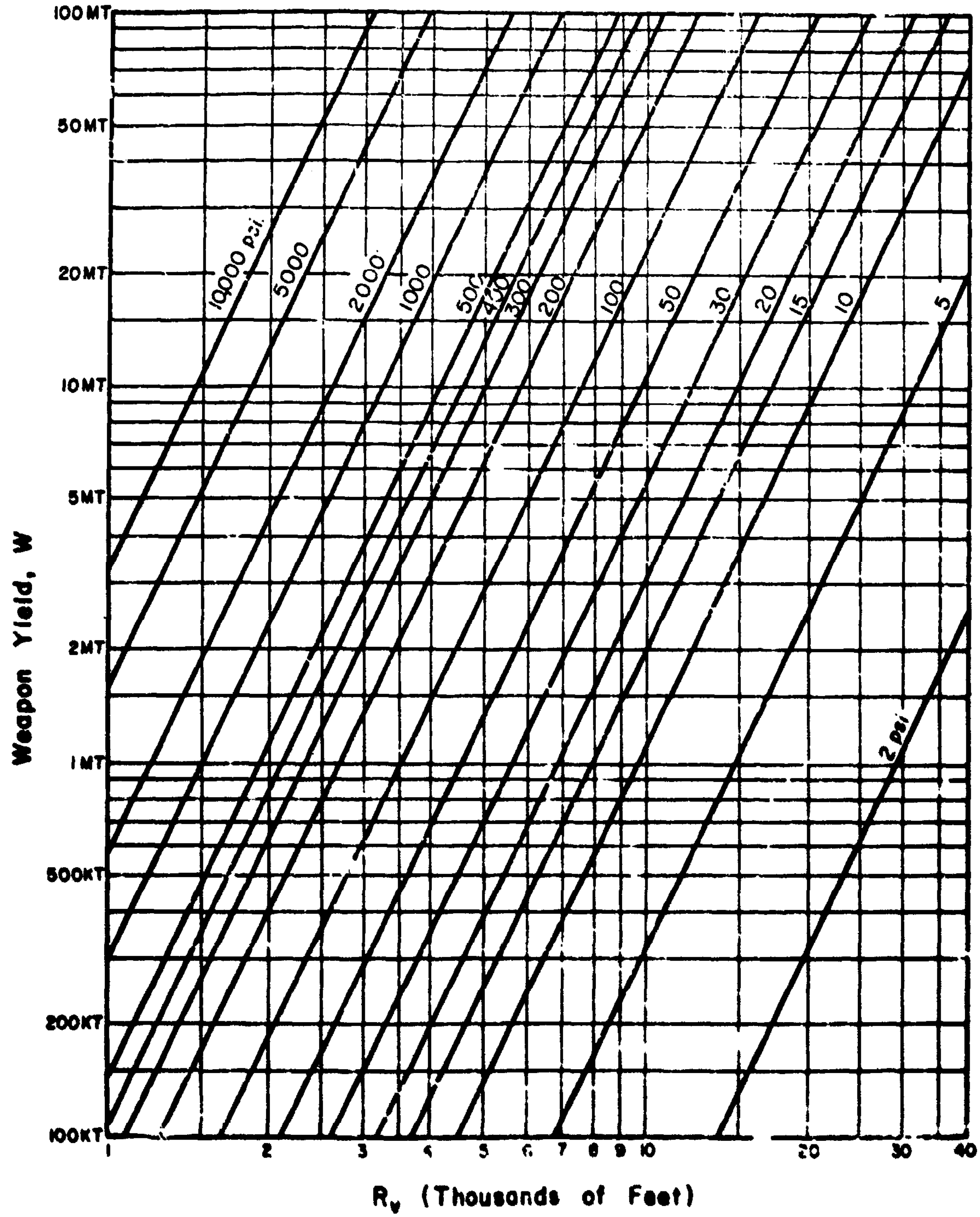


FIG. 2-7 WEAPON YIELD VERSUS VULNERABILITY RADIUS FOR VARIOUS SIDE-ON OVERPRESSURES AT GROUND LEVEL SURFACE BURST — SEA LEVEL.

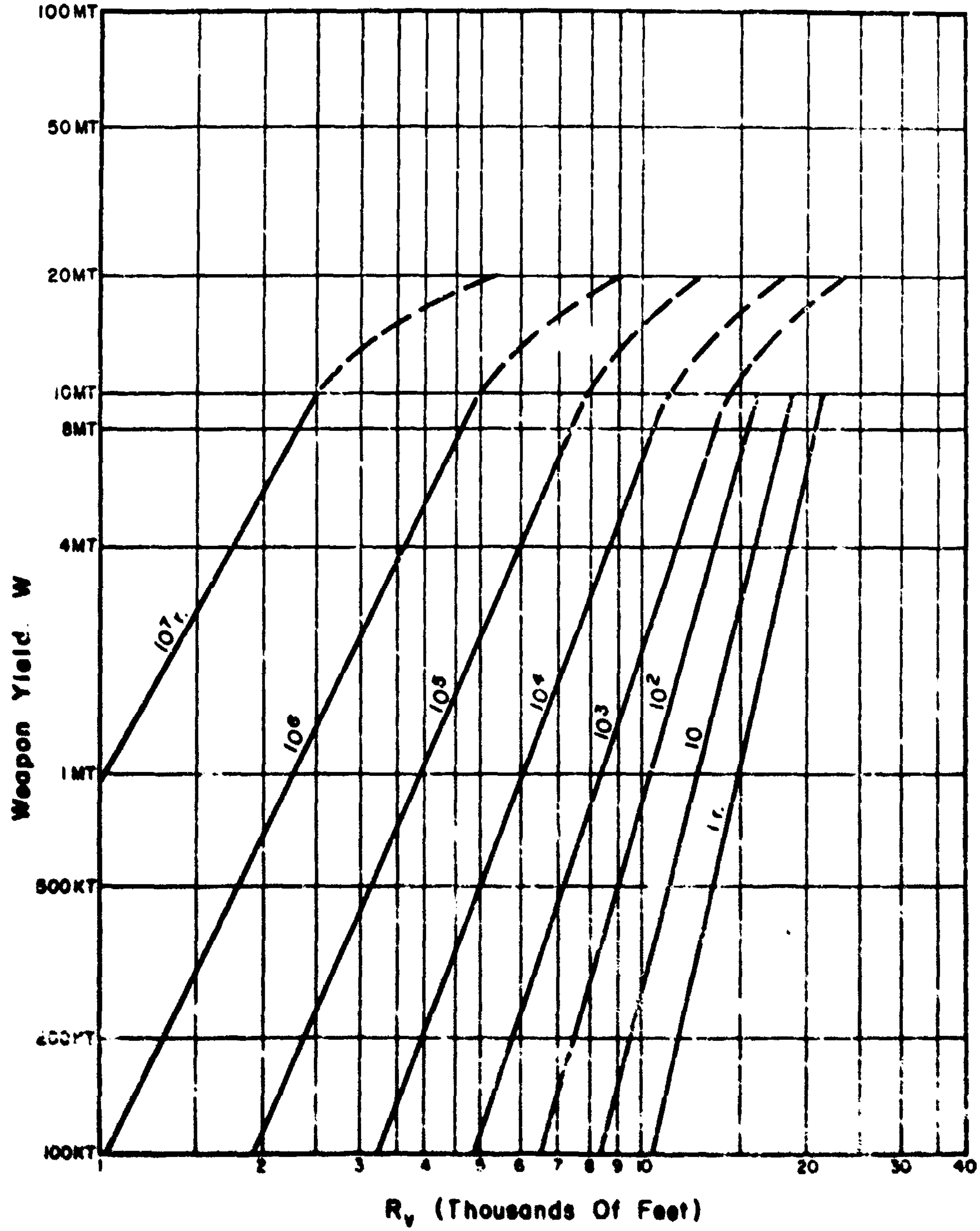


FIG. 2-8 WEAPON YIELD VERSUS VULNERABILITY RADIUS FOR VARIOUS LEVELS OF INITIAL GAMMA RADIATION — SURFACE BURST

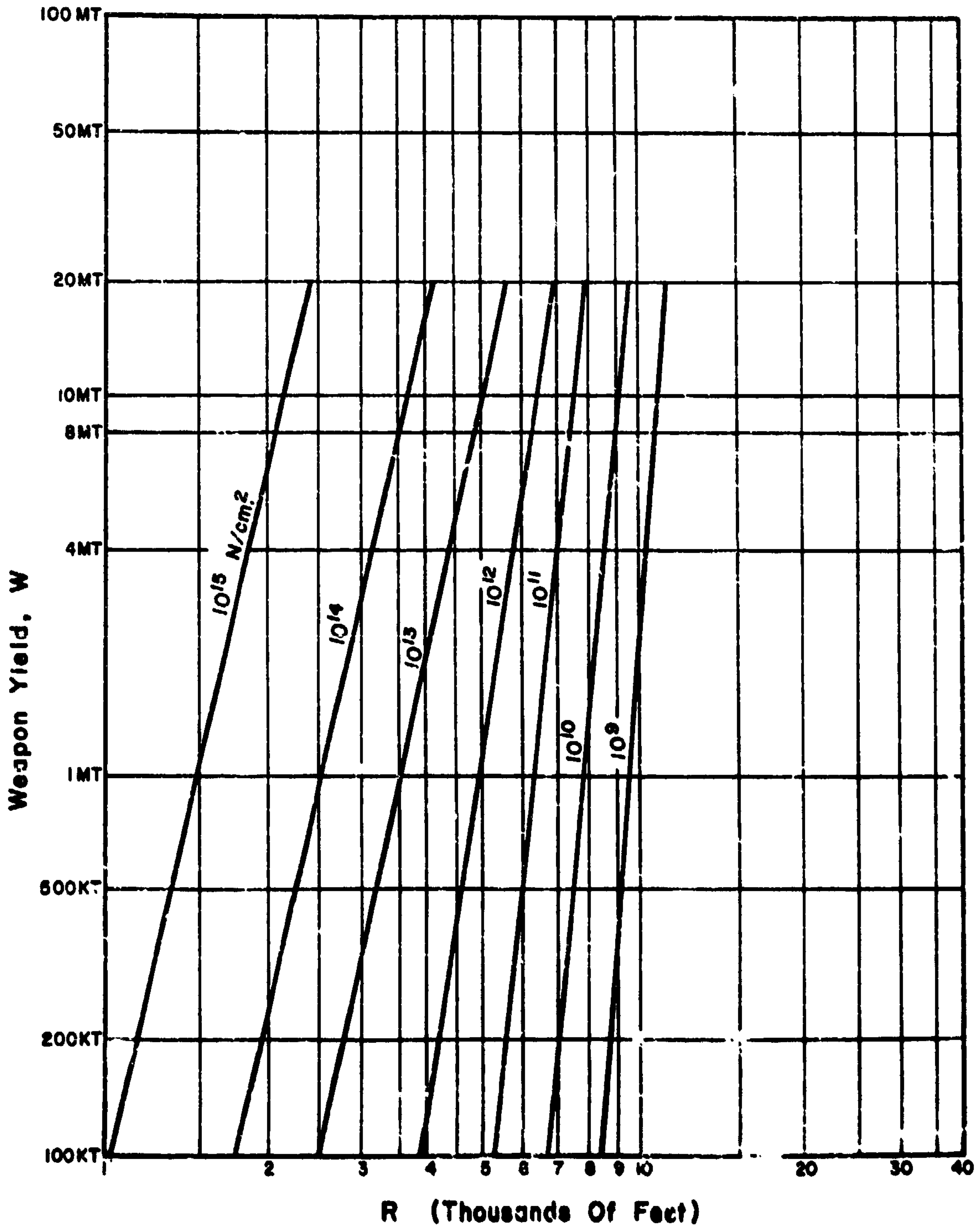
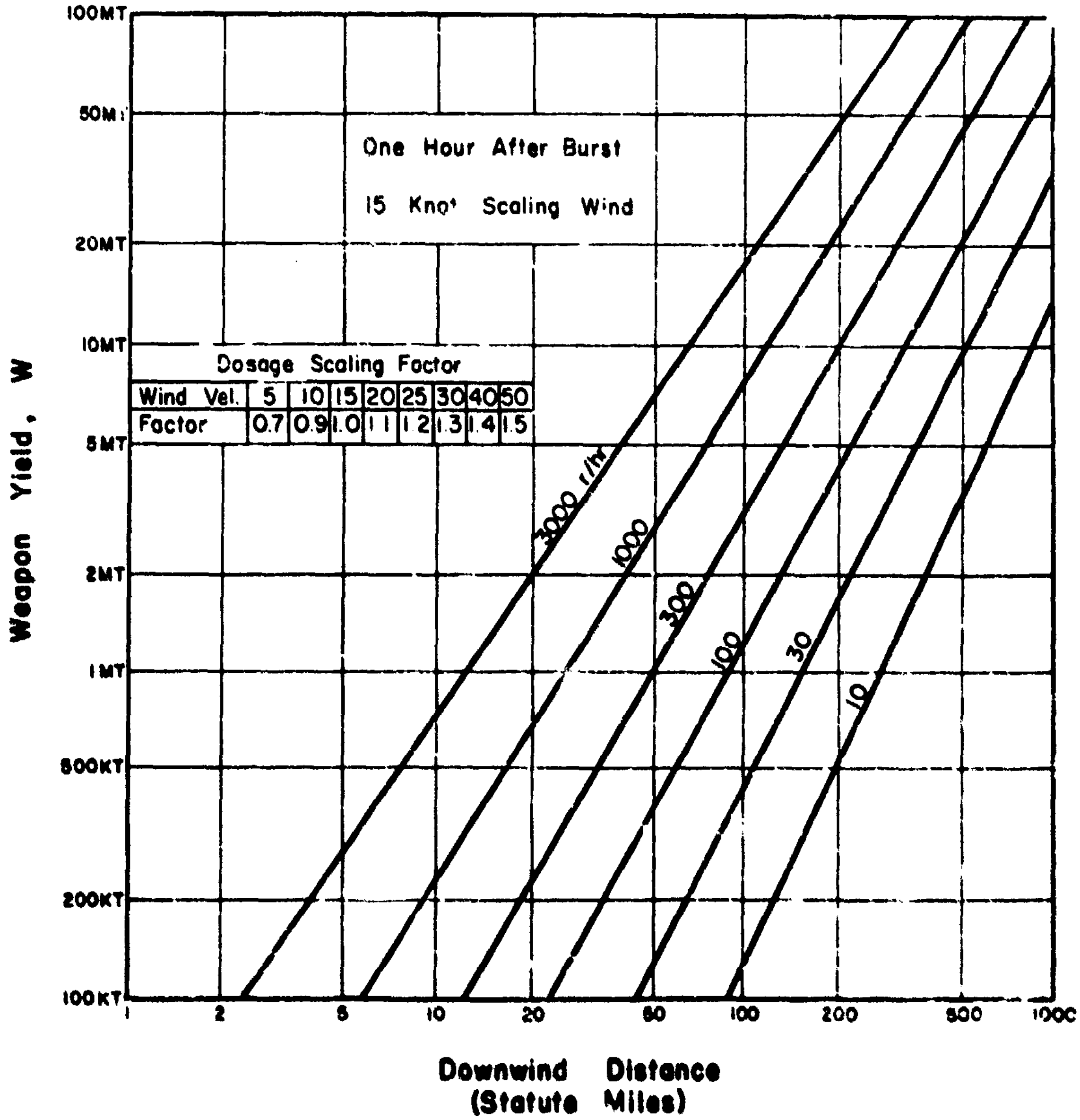


FIG. 2-9 WEAPON YIELD VERSUS VULNERABILITY RADIUS FOR VARIOUS LEVELS OF NEUTRON FLUX

OASD (P&I) MAY '61

NEWMARK, HANSEN & ASSOCIATES

Note: At Crosswind Distances Corresponding To The 3000 r/hr Downwind Distances, The Fallout Is Less T'an 10 r/hr



Dosage Scaling Factor

Wind Vel.	5	10	15	20	25	30	40	50
Factor	0.7	0.9	1.0	1.1	1.2	1.3	1.4	1.5

FIG. 2-10 WEAPON YIELD VERSUS DOWNWIND DISTANCE FOR VARIOUS LEVELS OF FALLOUT INTENSITY (1 HOUR REFERENCE DOSE)

OASD (P&I)
8052 DEC.'59—MAY '61

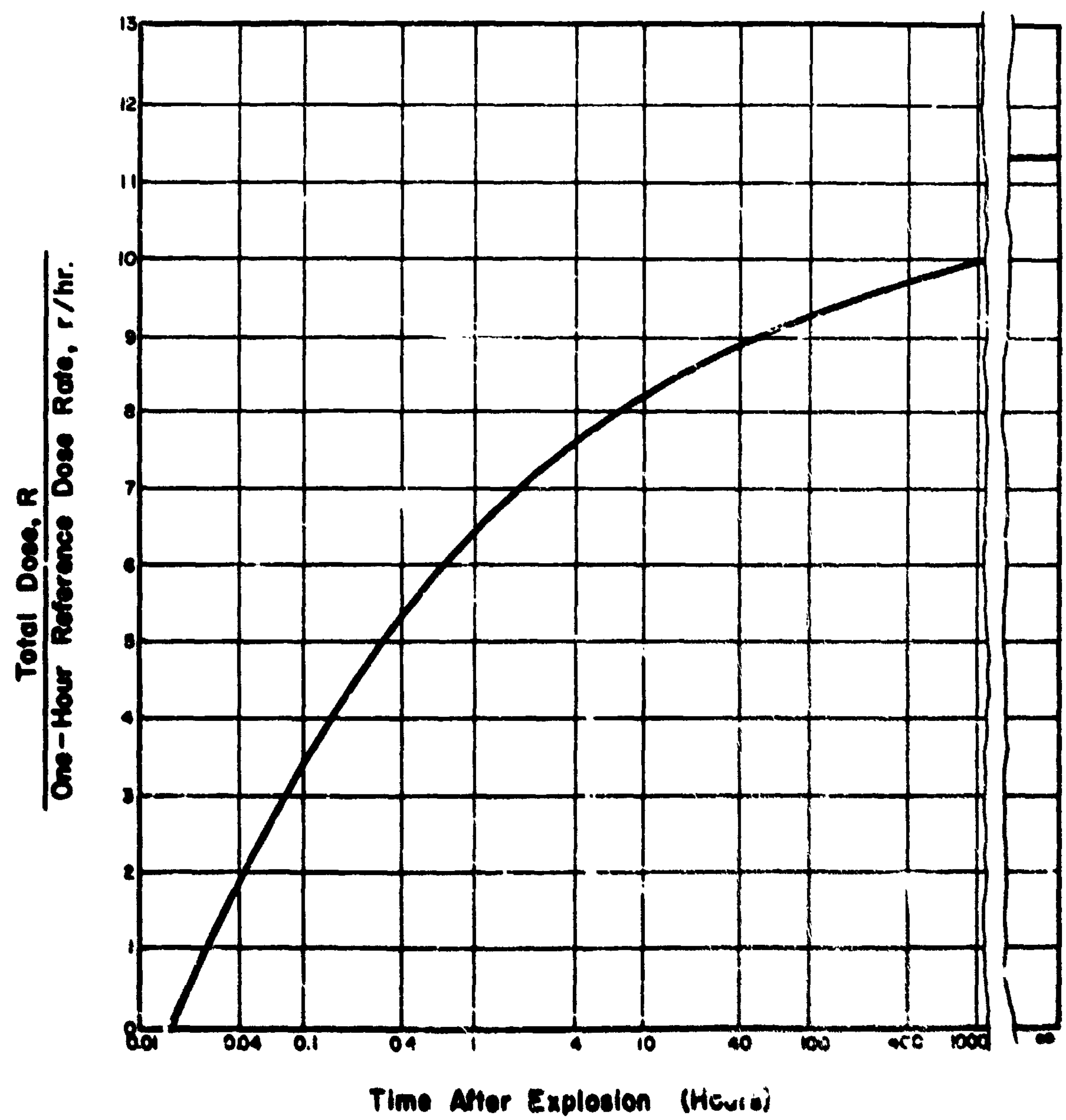


FIG. 2-11 ACCUMULATED TOTAL DOSE OF RESIDUAL RADIATION FROM FISSION PRODUCTS FROM 1 MINUTE AFTER THE EXPLOSION

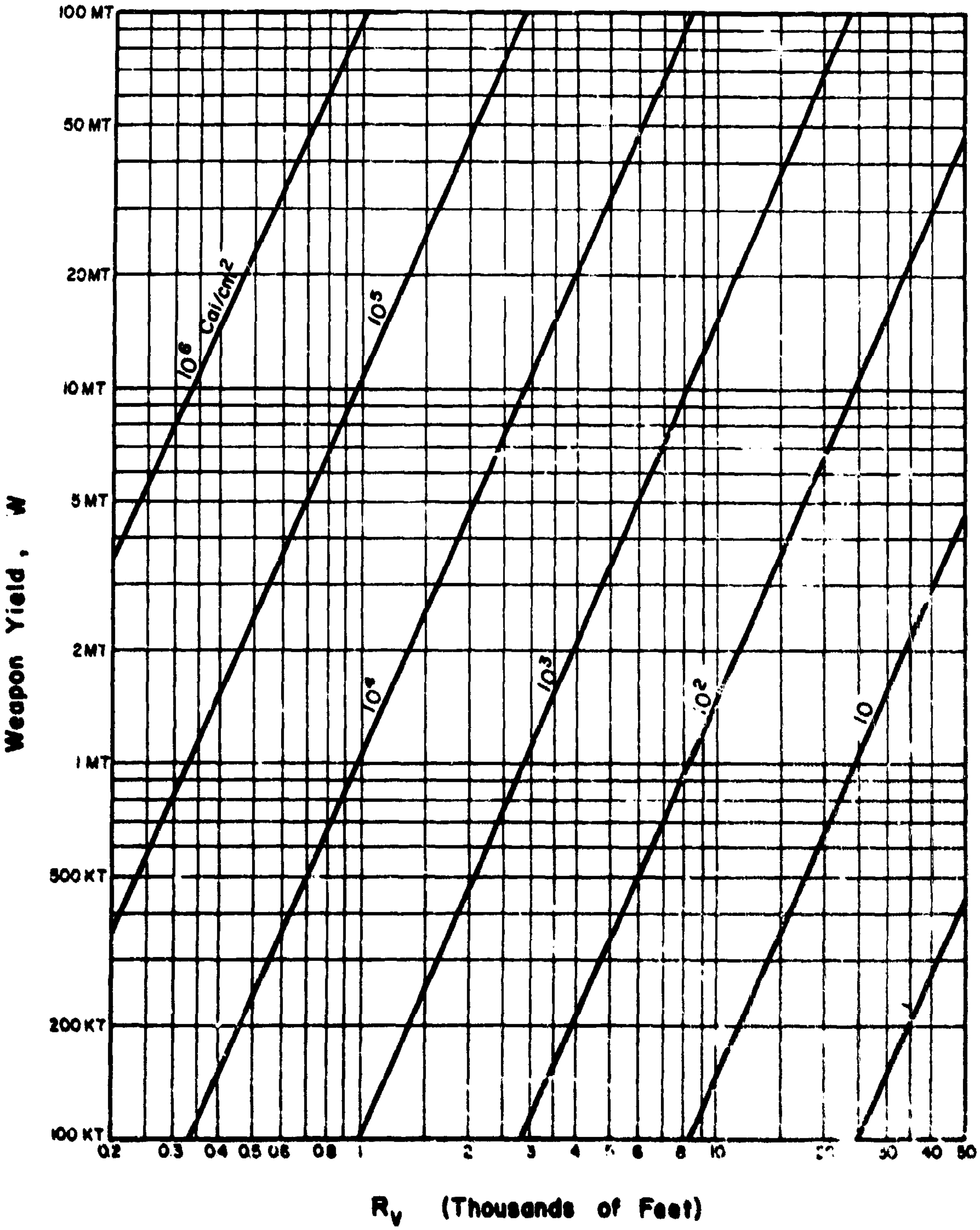


FIG. 2-12 WEAPON YIELD VERSUS VULNERABILITY RADIUS FOR VARIOUS LEVELS OF THERMAL RADIATION EXPOSURE — SURFACE BURST, VISIBILITY = 50 MILES

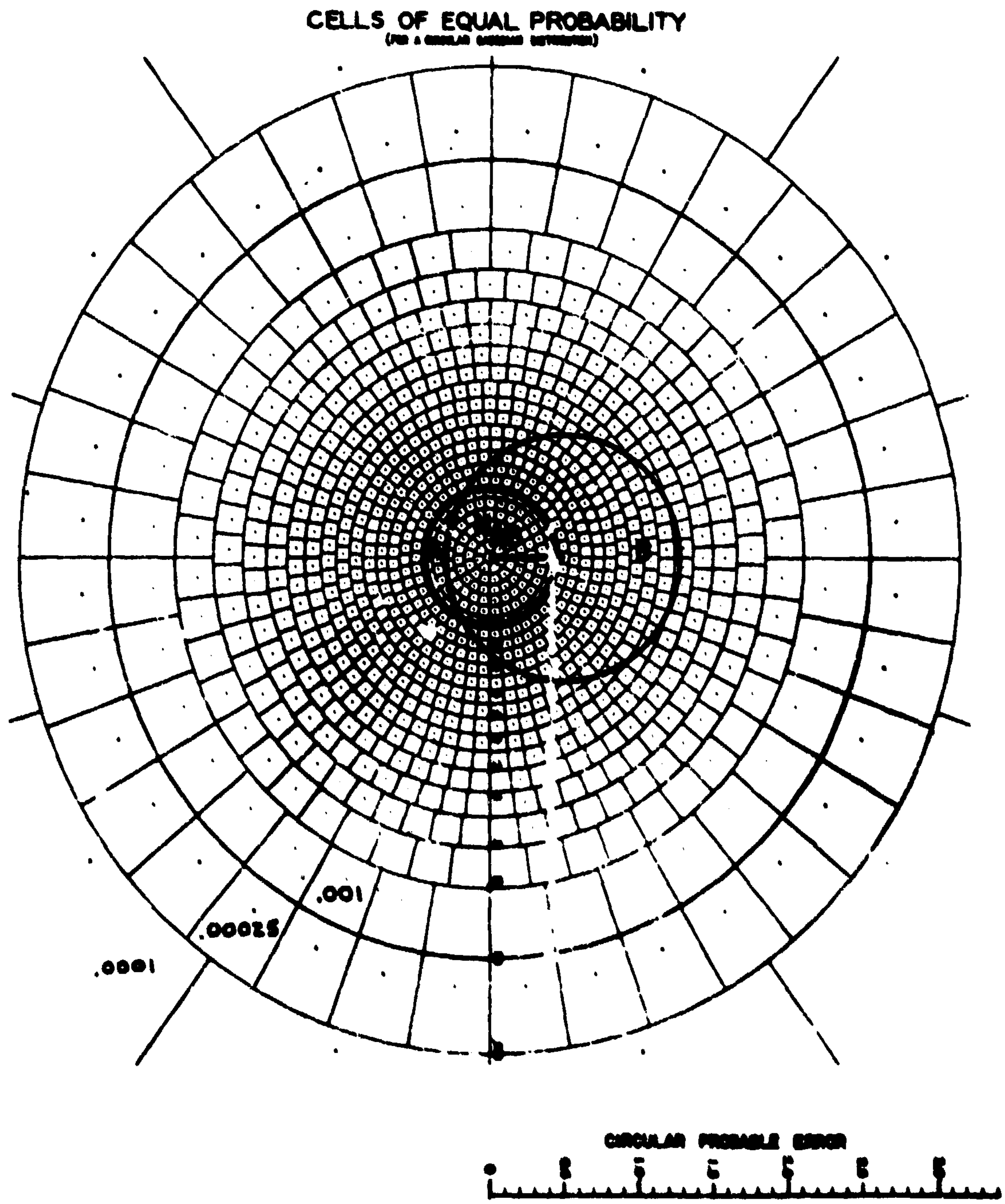


FIG. 2-13 SOLUTION OF EXAMPLE 3

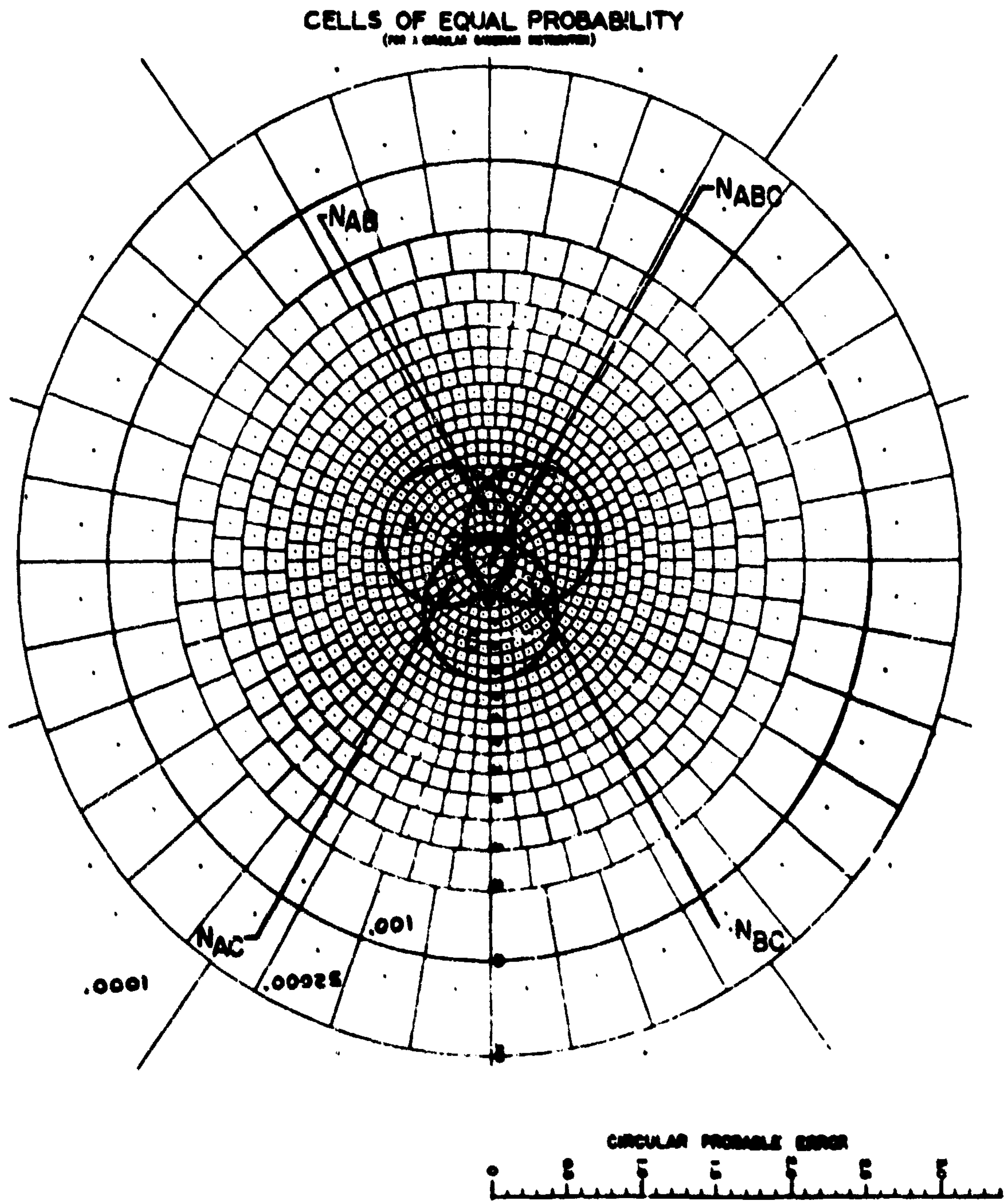
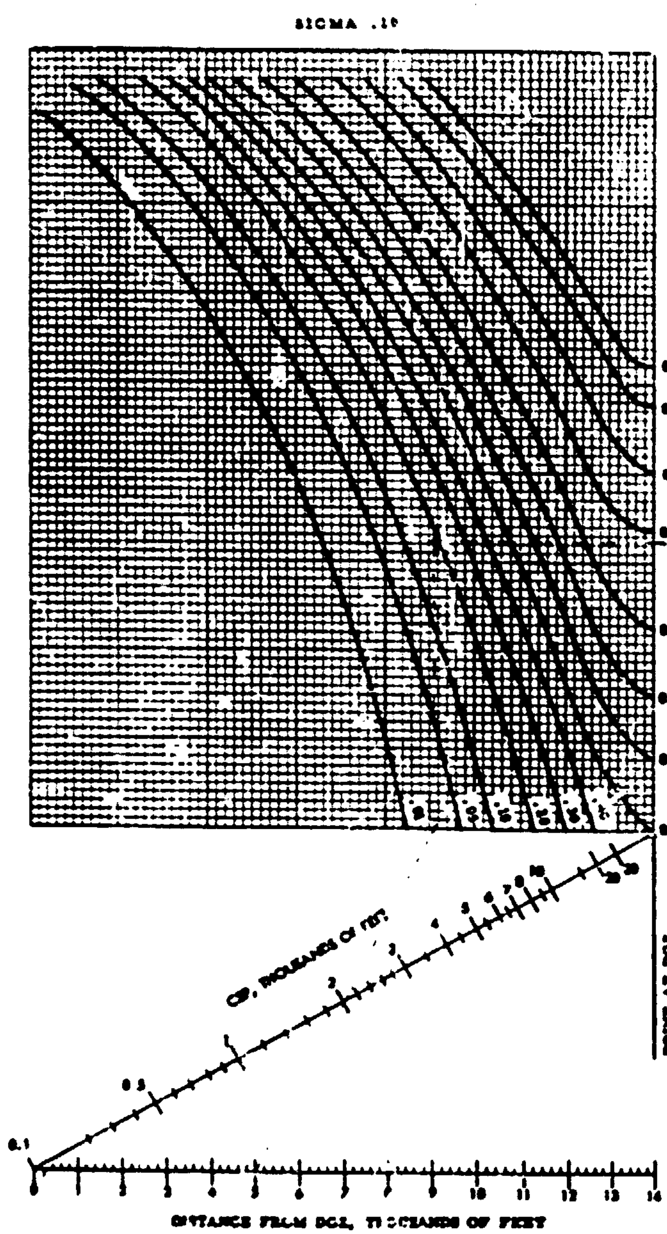
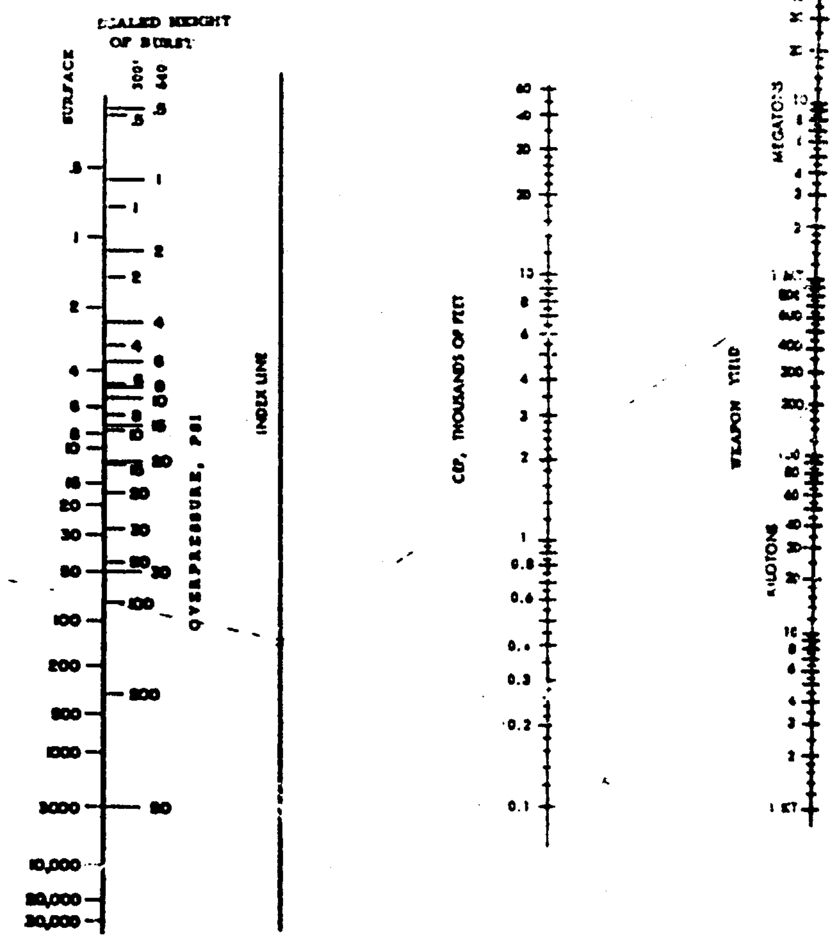


FIG. 2-14 SOLUTION OF EXAMPLE 6

PROBABILITY OF AT LEAST A CERTAIN OVERPRESSURE AT A POINT SOME DISTANCE FROM THE DGZ





SAMPLE PROBLEM

GIVEN: A point of interest 3000 ft from a DGZ. Weapon yield of 1 MT burst at a scaled HOB = 300 ft. CEP = 2000 ft

REQUIRED: Determine the probability of obtaining at least 100 psi overpressure at the point of interest.

SOLUTION: A. Draw a line from the 3000 ft point on the Distance from DGZ scale through the 2000 ft point on the CEP scale, intersecting the horizontal axis of the graph. Draw a vertical line through the graph.

B. Draw a line from the 1 MT point on the Weapon Yield scale through the 2000 ft point on the CEP scale, intersecting the Index Line.

C. From this point on the Index Line, draw a line through the 100 psi point on the 300' HOB scale, intersecting the vertical axis on the graph. Draw a horizontal line through the graph. At the intersection of this horizontal line with the previously drawn vertical line, read: probability = 0.18

NOTE: When the point of interest is located at the DGZ (Distance = 0), step "A" of the solution can be omitted. For this case, the probability is read on the vertical axis of the graph.

Prepared by [illegible], March [illegible]

FIG. 2-15 PROBABILITY OF AT LEAST A CERTAIN OVERPRESSURE AT A POINT SOME DISTANCE FROM THE DGZ.

NEWMARK, HANSEN & ASSOCIATES

OA [illegible] [illegible]
SDGZ MAY [illegible]

SECTION 5. DESIGN ANALYSIS

SECTION 5. DESIGN ANALYSIS

5.1 INTRODUCTION

5.1.1 Structural and Environmental Considerations. In planning a facility to resist the effects of nuclear weapons, consideration must be given to the following factors: (1) the immediate nuclear and thermal radiation; (2) the shock transmitted through the ground; (3) the air blast forces; (4) the radiation from fallout and the induced radiation resulting from the immediate neutron effects on materials.

The levels of radiation intensity and of shock and blast which an installation must resist depend on a number of factors over which the designer has no control. These include: the size of weapon and the distance from the target at which it is detonated, or the overpressure level which is produced at the installation by the weapon considered; the number of weapons which are successfully detonated in the vicinity of the installation; and the over-all pattern of the attack, which may bring residual radiation from fallout to a target which is not near any nuclear burst. To contend against these factors one may select the ground environment, the structural mass and strength of the elements of his installation, and the arrangement and multiplicity or duplication of the elements in a pattern which will give the necessary probability of survival. These factors influence the effects of the blast and radiation on the structure.

In the light of the discussions in SECTION 1 and SECTION 2 conditions are chosen which the structure must resist to give the appropriate probability of survival consistent with the importance of the facility. The tentative first conclusions as to the design parameters may have to be modified in accordance with operational considerations as described in SECTION 3. Guidance for determining survival criteria for personnel and materiel may be found in SECTION 4. In the light of these various discussions a tentative selection must finally be made of the structural type and of the design parameters, which can be stated generally in terms of the overpressure level and the radiation intensity levels, both from immediate and residual radiation, and the level of earth shock, for which each element of the facility must be designed. It is the primary purpose of this chapter to describe in detail how a design can be reviewed to investigate its suitability for the particular purpose intended.

It will ordinarily be necessary to consider structures of alternative types and to compare their relative costs in order to make the best selection for the particular item. In some cases, particularly for very high probabilities of survival, the cost may be correspondingly large. In these instances one may wish to consider the possibility of reducing these costs by a combined program of dispersal and hardening to a somewhat lower level to give the same probability of survival. The procedures described in SECTION 2 are available for this purpose.

In the various aspects of this discussion consideration is given to methods which are rapid but which are sufficiently accurate for all practical purposes, consistent with the accuracies of the other assumptions entering into the problem.

5.1.2 Structural Design. The purpose of this section is to describe the structural resistence required, and the methods for determining it, for various levels of overpressure and sizes of nuclear weapons. For a variety of structures and structural elements, charts are given for the review of the structure or element. The resisting capacities can be determined directly when a span or other controlling dimensions of a structure, the structural materials, and the range in the behavior of the structure for which the design is to be prepared, are given. Illustrative examples are given of the use of the charts.

In unusual cases for which charts are not given, the procedures in APPENDIX 5D may be followed. In general, consideration must be given to insure resistance against overturning and sliding of aboveground structures.

5.1.3 Mechanical, Electrical and Utility Design. In a hardened facility, the mechanical, electrical and other utilities equipment is conventional in design although there may be requirements for its shock mounting (See Para. 4.3). In general the principal difference in this equipment is in the nature and size of loads imposed by the hardened environment (particularly during buttoned-up periods) and the corresponding size of the equipment. These are discussed in Para. 5.7.

5.2 STRUCTURAL DESIGN AND REVIEW

5.2.1 Selection of Design Parameters. Design charts are given in APPENDIX 5A which may be used to determine the approximate size required for various structural elements such as slabs, beams, arches, domes, columns, footings, or frames for any level of overpressure. Use of these charts requires the definition of several parameters. These parameters are discussed briefly in this section.

The charts may be used to determine the resistance of elements of a given size in terms of the overpressure which would produce the desired degree of permanent deflection of such elements. It is assumed that the type and geometry of the structure or structural element has previously been determined. Most of the data are given in terms of the maximum pressure p_m to which the element is subjected. This is the peak side-on overpressure for aboveground roofs, the peak pressure attenuated with depth below ground surface for buried structures or the reflected pressure for aboveground slabs or walls. In all cases the charts include the effect of rapid application of load. Thus, the resistances given are dynamic rather than static. The relation of the reflected pressure p_r to the peak side-on overpressure p_{so} is given in Eq. (5D-1) in APPENDIX 5D.

The attenuation with depth of the vertical pressures on horizontal surfaces below ground can be obtained from Eq. (5-15) since the attenuation of overpressure with depth is of the same nature and magnitude as the attenuation of vertical velocity with depth. Factors for the horizontal pressure on vertical surfaces below ground are given in Para. 5.3.3.

The loads acting on underground silos and tunnels are discussed in Para. 5.3.

The charts for aboveground building frames, arches, and domes are given in terms of the peak side-on overpressure p_{so} instead of the maximum pressure p_m. These charts take into account the reflected pressure and the duration implicitly.

All charts are based on the assumption that the duration of the positive overpressure phase is long compared with the natural period of vibration of the element. For the structural forms considered this involves little error for megaton size weapons. For smaller weapons the charts would give conservative results but normally the error would not be serious. Other approximations are made in certain of the charts (see Para. 5A.3), and more refined methods which may be necessary in some cases may be applied by use of the general fundamental relations given in APPENDIX 5B and APPENDIX 5D.

All of the charts are concerned with the required structural resistance and do not consider the requirements for radiation protection, which subject is treated in Para. 5.5.

In order to determine the required size of elements, it is necessary to use the dynamic strengths of the materials, for example f'_{dc}, the dynamic compressive strength of concrete, and f_{dy}, the dynamic yield strength of steel. The magnitudes of these dynamic strengths, taking into account increase in strength due to the rapid rate of loading, have been included in APPENDIX 5B.

In the design of reinforced concrete elements it is necessary to select a steel percentage. In order to insure maximum energy absorption, it is recommended that where possible no more than 2 percent of flexural steel be used. The net flexural steel, which is the difference between the percentage of steel on the tensile side and that on the compressive side of the flexural member, should be kept less than 1.5 percent. Greater percentages of flexural steel may give failures of concrete in compression of a brittle character with considerably less energy absorption than would be obtained with a smaller percentage of steel. However, in unusual circumstances it is permissible to increase the net percentage to approximately $40\ f'_{dc}/f_{dy}$ provided that a low ductility ratio is used in the design, and provided that compressive steel is used. At the other extreme, it is imperative that at least 0.25 percent tensile reinforcement be provided in flexural members. Lesser percentages of tensile steel may result in brittle behavior of the member. However, in unusual circumstances the tensile steel percentage may be reduced to $2\ f'_{dc}/f_{dy}$.

When web reinforcement is required in a flexural member of reinforced concrete, at least 0.25 percent must be provided. Otherwise brittle behavior may develop. Because of the tendency toward rebound only vertical stirrups should be considered effective as web reinforcement.

The ductility ratio is a measure of the amount of plastic deformation permitted in the member or element. The ductility ratio, defined by the symbol μ, is the ratio of maximum deflection to the elastic limit or yield deflection. Charts are included for values of μ of 1.3 and 3.0. The smaller value of μ implies only slight damage to the structure since the permanent deflection is only 30 percent of the yield deflection, and the latter is normally small. This value should be used in designs when the functional requirements of the elements do not permit sizable deflections. The smaller value of μ, namely 1.3, is used also in the case of compression elements such as arches and domes, where it is doubtful that a great deal of plastic deformation can be mobilized without serious permanent damage. The higher value of 3.0 implies larger permanent deflections but not collapse of the element of structure.

Although considerably larger values of the ductility factor can be used in certain types of members, generally for the long duration loadings which are associated with megaton range weapons the difference in the strengths for a larger value of μ compared with the strength for value of μ of 3.0 is relatively small. The use of the lower value of ductility factor takes account of a number of uncertainties, such as those involved in the estimation of the properties of the material, the neglect of higher degrees of freedom, the precise shape of the loading and unloading curve, and similar matters.

In the case of reinforced concrete elements, charts are given for the various possible modes of failure including flexure, pure shear, and diagonal tension. In such cases all charts are read and the most severe failure criterion is used. The design charts for shear and diagonal tension are necessarily based largely on results obtained from extensive tests of beams and slabs of normal proportions; that is, with depth-span ratios less than about 0.2. The extension of these results to cover much deeper sections is done with some uncertainty. However, a limited number of very recent tests of deep reinforced concrete beams indicate that the charts may be somewhat conservative. Unfortunately no more reliable criteria are now available.

5.2.2 <u>Structural Details</u>. Since structural elements in a hardened structure are required to develop their full plastic strengths, particular attention must be paid to what are ordinarily considered details as discussed below for concrete and steel construction. It is emphasized, however, that these so-called details are extremely important in developing dynamic resistance.

Reversals of stress and reaction direction may occur. Accordingly, unless otherwise specified by analysis (see APPENDIX 5D) all members should be designed to have rebound strengths of at least 25 percent of the normal design strength specified by the blast loading.

While not strictly a structural detail it should be noted that special care is needed in mounting fixtures (such as lighting fixtures) in hardened structures. The structure can be subjected to considerable motions when subjected to an air blast wave and potential secondary missiles (even pictures hanging on walls) should be avoided.

a. Concrete Construction. Reinforced concrete is an excellent material for hardened construction. However, strict attention must be paid to details in order to assure continuity, ductility, and resistance to loads in either direction. Thus continuity of reinforcement by adequate lapping or welding is desirable, but welding may be difficult for certain steels. Diagonal tension reinforcement, which usually is required in hardened construction, generally should be perpendicular to the member axis because inclined bars form planes of weakness under the condition of shear reversal. In those instances where inclined bars are used to increase the shear resistance, additional vertical bars should be used to resist diagonal tension along the planes of weakness (under reversed loading) formed by the diagonal bars. Doubly-reinforced members with the reinforcing adequately tied have much more ductility than singly reinforced members and accordingly offer great advantage for hardened construction. Joints are particularly important. They should be detailed and fabricated in a way which will insure ductile behavior of the completed element. Further, the ultimate strength of the least strong connecting element should be developed in the joint, if at all practicable. In no case shall the amount of reinforcing used on any face of a beam or slab exceed 2 percent of the cross-sectional area of the element, in order to avoid brittle behavior. For doubly reinforced sections the reinforcing index q is given by

$$q = \frac{(\bar{p} - \bar{p}')}{f'_{dc}} f_{dy}$$

where $\bar{p}$ = ratio of tension-steel area to concrete area, bd, and
$\bar{p}'$ = ratio of compression-steel area to concrete area, bd.

As for singly reinforced sections, an index of approximately 0.40 divides underreinforced (ductile-behavior) sections from overreinforced (brittle-behavior) sections. Overreinforced members fail by crushing of compression concrete without yielding of tension steel. Such failures occur with little or no warning and obviously are not desirable particularly in hardened construction. Further, if other than billet steel bars of structural or intermediate grade are used, particular attention must be paid to avoid brittle behavior.

When heavy concentrations of longitudinal steel bars are required, particularly in the vicinity of lap splices, adequate transverse reinforcement (ties, stirrups, or simply transverse bars) should be provided to prevent bond failure by splitting of the concrete.

b. Steel Construction. Steel also can be used very economically for certain types of hardened construction. Arch or circular sections for underground construction, steel beams for composite construction, high strength

columns, and steel doors for personnel or equipment entrances are elements which may be more economically constructed of steel than of reinforced concrete.

Ductility, continuity, and development of full plastic strengths at joints are also recommended for steel construction. Steel members designed for maximum plastic resistance should be able to experience large deflections without reduction in load capacity. Generally such members will be stockier than in conventional designs. Recommendations of proportions which would minimize or avoid buckling problems are given in APPENDIX 5B.

In the design of vertical members of continuous frame construction, fixed column bases, if combined with a suitably strong foundation, will increase the plastic resistance of the entire frame. In addition, if the column top is restrained against rotation about both axes by members that frame in both directions, the plastic strength capacities of the column will be increased by reducing tendencies toward lateral buckling.

Connections should be designed so that ductile behavior should take place in the member. In corner connections it is particularly desirable to introduce diagonal stiffeners so that undue shear yield, causing large local angle-change between connecting members, will be prevented. Similarly, connections for tension members should develop the yield strength in the main body of the member.

Weldable steel should be specified and welded joints should provide maximum continuity of detail and a minimum of local stress concentration. In riveted and bolted joints, sheared edges and punched holes should be avoided and generous edge distances should be used.

5.2.3 Hardened Doors. The intended function of a hardened installation may be achieved or lost according to the attention that is given to the doors. It cannot be overemphasized that doors - particulary large doors - represent a major structural-mechanical design problem, and that door requirements often may influence the type or proportions of the main structure. Careful study of this problem is further necessitated by the fact that the total cost of a large door, including its mechanical and electrical components, can represent a significant fraction of the total cost of the hardened installation. It follows that door studies should be initiated at the outset of the over-all planning and design. As a corollary the reviewer of a proposed hardened installation should scrutinize the door provisions most carefully.

It should not be inferred that every door requires a detailed design study. On the contrary, small doors, particularly those which are not so located as to experience all of the weapons effects, should be standardized types whenever possible. Such small doors occur frequently throughout the field of hardened construction, and standardization is highly desirable to minimize costs and construction delays.

Paragraph 5.2.3.2 is comprised of a list of functional requirements which should be considered by the designer. This list should likewise be referred to by the reviewer in evaluating proposed types or arrangements of doors. Paragraph 5.2.3.3 is comprised of a list of important characteristics of the hardened door, with comments on the significance of each. Supplementary source material is provided by a list of pertinent references.

5.2.3.1 Types of Doors. In general, doors may be classed in accordance with their attitude, either horizontal, vertical or inclined, and with respect to their method of opening, sliding or rolling, hinged on one side or on both sides with a joint down the center. A third method of classification involves the configuration: Whether the door is flush with a surface, in a recess or outside a surface with the edges exposed.

There are advantages and disadvantages in each type of configuration, method of opening, or attitude. Horizontal doors have the advantage that they are subjected only to side-on overpressures whereas inclined or vertical doors may be subjected to the much higher reflected pressures. On the other hand, horizontal doors may have to be larger than vertical doors to provide entrance for certain items of equipment or personnel although they may have advantages in openings for missile silos where only a vertical entrance to or exit from the enclosure is needed.

Sliding flush doors have certain advantages in mechanical simplicity although some difficulties are presented with regard to the exposed free edge. Such a free edge is exposed to blast forces and drag pressures for which provision must be made in the supports of the door. Difficulties are also encountered in providing for seals against blast pressure and dust. Provision must be made for removal of debris either by the door itself as it slides forward or through auxiliary means, in order to permit opening of the door when the surrounding area is covered with the debris resulting from a close-in burst. Some of these disadvantages are overcome by doors which swing on hinges of the single or double leaf type. Such doors may be made to be practically self cleaning of debris, but generally require more careful attention to detail in the mechanical arrangements at the hinges than do sliding doors. Special provision must be made in double leaf doors for the sealing of the enclosure where the free edges meet. The relief of the hinges from blast loading which is usually necessary also presents mechanical problems which can be solved, although not simply. Doors which must remain open for operations in severe wind storms present additional problems. Flush sliding doors provide less resistance to winds and less turbulence in the region of the enclosure than do doors which stand up in the wind stream when they are open.

Vertical doors may also be hinged or may slide. Sliding vertical doors are usually supported at their bottom surface. Hinges for vertical doors are usually most conveniently provided at the bottom so that the door swings open as in a drawbridge, or at the top in which case the door may swing either outward or inward. However, heavy doors are difficult to swing from hinges at the top and to support when they are closed.

In addition to the door types discussed above there may be other unusual types of doors that offer special advantages for particular uses. For very high overpressures, doors may be used which are relatively thick and slide into place against a solid wall so that the door itself is not subjected to high stresses under blast conditions. When the door is slid to one side into a pocket, a right-angled entranceway is formed which may be adequate for personnel and for small vehicles but is generally not capable of being made adequate for large equipment or vehicles. Other types of closures may involve plugs of rock or earth which can be removed after a blast, but not quickly. Such closures would not ordinarily be adequate for installations which require a short reaction time. Other energy absorbing doors such as doors involving masses of water or mechanical energy absorbers may find particular uses. In general such doors involve a great deal more complexity than do simpler and more rugged although possibly more massive doors.

5.2.3.2 Functional Requirements.

a. Existence of Alternate Openings of Similar Function. Is the installation such that more than one opening is desired under day-to-day or attack conditions? If necessary can the installation function after an attack with less than the total number of doors operable? If so, can alternate openings be oriented to avoid or minimize the probability of full weapons effects at all locations?

b. Exposure. Is the opening at an exposed (i.e., surface) location or is it within a tunnel or other shielded location? In the latter case some weapons effects (thermal, radiation, reflection increments to shock pressure, dust and rubble) may not have to be taken into account in the door design.

c. Day-to-Day Function. Does the door have to operate fairly frequently, in day-to-day functioning of the installation? Only infrequently? Only rarely (as a check on readiness)?

d. Time Available for Operations. What is the maximum time that can be permitted for door opening and for door closing under attack conditions? In day-to-day operations?

e. Number of Post-Attack Operations. Is it only required that the door survive one attack, or must the closing-survival-opening cycle be guaranteed through several successive attacks?

f. Orientation Requirements. Is the purpose of the opening such that the door must be horizontal? Or vertical? Or can the orientation be selected to minimize door loading without regard to function?

g. Susceptibility of Installation Contents to Various Weapons Effects. For the (human or material) contents of the installation is protection required against all attack effects (blast, heat, radiation, chemical and biological contamination)? Are the contents insensitive to one or more of these effects? Is the occurrence of one or more of these effects deemed improbable?

h. Required Size and Shape of Opening. Although size and shape of opening are abvious criteria, it seems necessary to emphasize that an underestimate of these requirements may impair the function, and an overestimate will needlessly increase costs.

i. Operational Limits on Position of Opened Door. To avoid interference with operations are there limitations on positions of the opened door?

5.2.3.3 Important Door Characteristics.

a. Strength and Stiffness. If some plastic deformation can be accepted this will reduce the strength required to resist the given blast pressures. The extent to which such deformation can be tolerated depends not only upon the failure mode of the door but also upon the influence of distortion on subsequent operation. In particular, excessive distortion may jam the door so that it cannot be opened, or cannot subsequently be closed; may break joint seals or make a tight seal in subsequent closings impossible.

It should be noted that the door and its supports may be subject to force reversal, i.e., forces opposite in direction to the blast pressure. Such forces may result from ground accelerations or from the elastic rebound of the door.

b. Weight. The required thickness may be governed by required resistance to radiation effects, in which case the weight may not be subject to measurable control by skillful structural design. When thickness is governed by blast loading, however, the weight may vary widely with door form (dome, slab, etc.), method of support (full perimeter, two-edge, points, etc.), materials (steel, concrete, or combination), and internal structure (solid, cored, sandwich).

The size and cost of mechanical components of large doors may be very sensitive to the weight of door structure. This would be particularly true in those door types for which the (unbalanced) mass must be lifted. It is to be noted that the cost of mechanical-electrical components (trunnions, rollers, jacks, power cylinders, linkages, gears, motors, tracks, etc.) may be a large fraction of the total cost. Accordingly, an increase in cost of door structure to reduce weight may reduce the over-all cost of a large door. In addition, reduction in weight of door structure may be significant in terms of reduced power requirements.

c. Shape of Exposed Surface. In many cases the most appropriate solution of the door structure is a flat slab of concrete or steel, and the major exposed surface is a large flat plane. For more effective use of the material, dome types also have been considered. In the latter type much of the inherent strength advantage may be lost because of the more severe loading associated with reflection and drag effects on the dome surface, in contrast with loading on the plane surface of the slab type. This difference in loading is particularly pronounced when the slab type door is recessed to make its outer surface flush with the outer surface of the main

structure. This arrangement is particularly effective for horizontal doors (vertical entrance) designed for resistance to the blast effects of a surface burst. In this situation the two advantages achieved by a flush surface (with adequate sealing of the perimeter joint) are: elimination of reflection effects in the blast loading; elimination of forces in the door plane due to drag and due to blast pressure on the vertical edges. When the door must be vertical (horizontal entrance) the dome shape is not at a disadvantage with respect to forces normal to the plane of the protected opening: however, it is less satisfactory than the recessed slab type with respect to forces in the plane of the opening.

d. Degree of Protection Afforded Door Mechanism. Exposed parts of a door mechanism may be damaged, and the door rendered inoperable by heat, fragment missiles, rubble, or dust. Whether one or more of these hazards must be considered depends upon door location (at surface or well within a tunnel or other shielding), nature of adjacent terrain surface, proximity to other buildings or equipment which might furnish fragment missiles, and assumed weapon size and DGZ.

When these hazards are present preference should be given to designs which place all of the door mechanism within the protected space. In general such complete protection is feasible. In the rare case of doors so enormous that a practical method of operation must involve rolling on exterior tracks the protection of these elements may be a major problem. Fortunately this situation will occur only rarely, if ever.

Protection of the door mechanism involves not only the direct weapons effects listed above but also the effects of the very large forces transmitted by the door structure, and the distortions and motions which the door structure may experience. Door forces during blast are very much larger than dead weight forces. Thus while the mechanism can be designed to work against the latter it is not feasible to provide even static resistance to the former. For this reason, and because resistance to distortion and relative motion requires large mechanism forces, the operating mechanism should be isolated from these forces and motions when the door is in the closed position. Support for the closed door structure should be independent of the trunnions, rollers, struts, and other elements of the door mechanism. In addition the mechanism must, of course, be resistant to the effects of ground shock.

e. Reliability of Mechanism Power Source. Small blast valves and doors may incorporate integral power sources in the form of compressed springs, explosive cartridges, and power cylinders. Because of the small door mass in these cases the power requirements are small and emergency hand operation often may be provided.

Large doors often require very large power expenditures for short periods of time. For day-to-day operations this power can be drawn from a central source, even a source exterior to the installation. In such cases, however, a parallel standby power source always should be provided within the protected space. Consideration should be given to the use of hydraulic-pneumatic power systems which have the advantage of requiring relatively small electric power input to a pressure accumulator.

In some cases it may be possible to utilize counterweighting to reduce the power required for door operation. In other cases gravity forces may be employed to open (or close) the door without power input. If gravity is used to facilitate opening, the opening time can be substantially reduced without increasing the power requirement.

f. Reliability of Warning and Triggering Devices. It is essential that remote warning devices and circuits be provided to initiate door closure, and that these provisions be matched with the door closure time. Consideration should be given to "fail safe" circuitry which will initiate door closure in the event of failure or malfunction of devices or circuitry.

5.2.3.4. Partial Bibliography on Doors.

(1) "Designing Entrances for Protective Shelters", Cdr. L. N. Saunders, CEC, USN, BUDOCKS Technical Digest No. 80, Sept. 1957

(2) "A Door Design to Protect Large Aircraft Against High Overpressures", Sargent White, Phys. Vuln. Div., Directorate of Intelligence, USAF

(3) Course Outline for Course on Atomic Defense Engineering, Port Hueneme, Sept. 1958

(4) "WS 107 A-2, Technical Facilities, ICBM-Base T-1, Concepts", prepared by DMJM for AFBMD (Secret)

(5) "Evaluation Report for IOC Operational Main Closure, WS 107 A-2 Launcher System", prepared by AMF under Contract AF 04(647)-138 (Secret)

(6) "Proceedings of the Symposium on Protective Construction", Sept. 21-22, 1954, Washington, D. C., Office of Chief Eng., U. S. Army (Secret)

(7) "WS 107 A-2, Technical Facilities, ICBM Base T-1, Preliminary Basis for Design and Outline Sepcifications", prepared by DMJM for Corps of Engineers, U. S. Army (Secret)

(8) "A Protective Alert Shelter for Strategic Air Command", Associated Research Design, Contract AF [illegible])-1636 (Secret)

(9) "Proceedings of the Symposium on Protective Construction (U)", RAND Corp., ASTIA Document AD 150659 (Secret)

5.3 DESIGN LOADINGS FOR SILOS AND TUNNELS

5.3.1 Arching for Dead Load and Live Load. Long shafts either vertical or horizontal present particular problems in design. Such shafts are encountered in tunnels connecting various structures in a complex, or in vertical shafts housing various kinds of equipment or providing access to structures. The loadings on shafts of this sort are affected greatly by the method of construction and by the properties of the soil. For this reason, general recommendations are given in this section for the design loadings to be used for various conditions that might be encountered in practice. These loadings are used with the design procedures and charts given in APPENDIX 5A arches.

In a body of soil that is undisturbed and does not have structures located in it, the situation is considered "at rest", and the vertical pressures due to the dead load of the soil are equal at any point to the weight of the column of soil above that point. The horizontal pressures are generally proportional to the vertical pressures, and the ratio is called the "Coefficient of earth pressure at rest". This coefficient is usually considerably greater than the coefficient of "active" earth pressure, which corresponds to the minimum lateral pressure required to keep the soil from sliding into an opening, but it is considerably less than the coefficient of "passive" earth pressure which corresponds to the maximum lateral force that can be developed.

In tunneling operations the soil is disturbed and movements take place during the construction. Because of these movements the pressure of the soil against horizontal and vertical surfaces is changed from the free-field conditions and is generally considerably reduced below the at-rest condition, but the lateral pressures may be somewhat higher than the active lateral pressures. The methods described herein for computing the pressures for horizontal or for vertical shafts are adapted with minor modifications from the procedures given in Ref. 4.

When additional load is placed on the surface of the ground, the pressures on underground structures depend on the coupling between the structure and the soil, and on the soil and structure properties as well. However, for blast loading, the soil strengths are somewhat higher than for dead loading or for long-time loading. Also, the structures are generally fairly stiff and cannot move as much as the movements that are permitted during construction operations. Consequently, the forces transmitted by the surface blast loading to the structure may be considerably higher, at least in lateral directions, than the dead load active earth pressures or nearly active earth pressures for which the structures are designed.

In the treatment which follows, primary attention is given to sand or to granular materials having an angle of internal friction. However, the same treatment may be used, with some slight degree of overconservatism, for cohesive materials provided that the cohesion is considered to be in effect something which can be replaced by an equivalent angle of internal friction.

5.3.2 <u>Dead Load Lateral Soil Pressures on Silos or Vertical Shafts.</u> Consider a vertical shaft of radius r in a granular material, shown in Fig. 5-1, where the pressure distribution is shown schematically on the wall of the shaft. The vertical pressure at a depth z is noted by the symbol p_z. Because of the movements during construction operations the lateral pressures may have values considerably below the lateral pressures "at rest", and there is a tendency for the lateral pressures to arch around the wall of the shaft. The recommended pressure distribution to be used for design of the structure for dead load is shown in Figs. 5-1 and 5-2. Figure 5-1 gives, as a function of the ratio of the depth z to the silo radius r; the ratio of the pressure p_z at depth z to the pressure at an infinite depth p_∞. The curve shown has the equation:

$$\frac{p_z}{p_\infty} = \frac{z/r}{z/r + 2.5} \tag{5-1}$$

The pressure at an infinite depth is shown in Fig. 5-2 in terms of the density or weight per unit of volume of the soil w and the radius of the silo r, as a function of the angle of internal friction Φ. Values of Φ below 30 degrees are not found for sand. Values of Φ for silt may range down to 25 degrees. The design curves are not applicable at all below 25 degrees.

As an indication of the way in which these figures can be used, there is shown in Fig. 5-3 the horizontal dead load pressures on a 50-ft. diameter silo for various angles of internal friction. The curves shown are plotted for a material of a density w = 120 lb per cubic ft., and the pressures are given in psi as a function of the depth from the surface in feet. There is given along each of the curves the pressure at an infinite depth computed from the coefficients in Fig. 5-2. It can be seen from Fig. 5-3 that for material having an angle of internal friction of 35 degrees, although the pressure at an infinite depth is 13.9 psi, the pressure at a depth of 150 ft. is only 10 psi and the pressure at 50 ft. is about 6.3 psi.

The calculations described are for essentially dry material. For undrained conditions and an impervious structure, the pressures of the water below the watertable must be considered. It is appropriate, under the conditions where water is present to reduce the weight of the material below the watertable to the submerged weight. However, in most cases the effect of the water is so much greater than that of the soil that it is usually reasonably accurate merely to add the horizontal pressures due to the water below the watertable.

5.3.3 <u>Lateral Live Load Soil Pressures on Silo.</u> If a large enough area were loaded uniformly with blast pressure, and the pressure lasted for a relatively long time, the loading situation would correspond almost to a uniform vertical pressure, independent of depth. Under such conditions, the horizontal pressures induced in an elastic material having a value of Poisson's ratio of $\bar{\mu}$ are equal to the following:

$$q = \frac{\bar{\mu} p}{1 - \bar{\mu}} \tag{5-2}$$

where q is the lateral pressure and p is the vertical pressure. The lateral pressure is a compressive force. The vertical pressure, and consequently the horizontal pressure, attenuate with depth as a result of the motion of the blast over the surface and the decay of the overpressure with time. As mentioned in Para. 5.2.1 this attenuation is like the attenuation of velocity in Eq. (5-15). For large yield weapons this attenuation may be neglected for the depths normally considered of interest.

If a vertical shaft were in existence without pressure being applied to it, and if the shaft were not deformable, the lateral pressure on the vertical walls of the shaft would also be equal to q. If, however, the material lining the shaft is compressible, the pressure on the shaft is reduced. This reduction is a function of the relative moduli of elasticity of the shaft material and the soil as well as the ratio of the thickness of the shaft to the radius. The magnitude of the value of Poisson's ratio $\bar{\mu}$ of the soil enters also. Designate by the symbol N the appropriate function of these quantities:

$$N = (1 + \bar{\mu}) \frac{ED}{\bar{E}r} \qquad (5\text{-}3)$$

where E is the modulus of elasticity of the lining, $\bar{E}$ the modulus of elasticity of the soil, and D the thickness of the lining. Under these conditions, and with the value of N defined as above, the maximum radial pressure on the lining is given by the equation:

$$p_h = \frac{N}{N + 1} q \quad . \qquad (5\text{-}4)$$

The circumferential compression in the soil, p_θ is given by the relation:

$$p_\theta = \frac{N + 2}{N + 1} q. \qquad (5\text{-}5)$$

These relationships may be used as approximations to give the live load lateral pressures on a vertical shaft wall. In general, the reduction in pressure produced by the compressibility of the liner is negligible. For example, if a shaft is considered with a thickness of two feet, a diameter of 40 ft., with a value of Poisson's ratio of 0.2, and with moduli of 4,200,000 psi for the shaft material, and 100,000 psi for the soil, a value for N of 5 is obtained. Then one finds a lateral compressive force on the shaft of 0.83q with a circumferential compression of 1.17q. For the value of Poisson's ratio quoted, the value of q is 0.25p, and consequently the lateral pressure on the shaft is 0.21p.

Because in general the value of Poisson's ratio for various kinds of soil is not known, it is suggested that the ratio of the horizontal to the vertical pressure for dynamic loading be taken as follows:

For cohesionless soil, damp or dry	1/4
For unsaturated cohesive soils of stiff consistency	1/3

For unsaturated cohesive soils of medium consistency	1/2
For unsaturated cohesive soils of soft consistency	3/4
For all saturated soils where the water table is at the surface	1

When the watertable is more than 30 ft. below the surface use the value for the unsaturated condition for the same type of soil. For intermediate levels of the watertable between 0 and 30 ft., interpolate linearly for the value of the lateral pressure coefficient for points below the watertable, and use the values in the preceding tabulation for points above the watertable.

5.3.4 Non-Uniformity of Circumferential Pressure. For a vertical shaft which intersects the ground surface, a non-uniform load develops around the shaft over the portion near the surface. This non-uniform load exists as long as the shock is enveloping the silo. To account for this non-uniform loading it is recommended that the shock be considered to be made up of two components over a depth below the surface equal to the diameter of the silo. One component is a uniform compression acting around the circumference; the other is a sinusoidally varying pressure consisting of four half-sine waves around the circumference, alternately inward and outward. The maximum amplitude of each component should be taken equal to one-half the peak side-on overpressure at the surface. The stresses caused by these two components of loading should be superimposed and the maximum stresses should not exceed those defined in APPENDIX 5B. The uniform component causes a "hoop compression" in the walls of the shaft. The sinusoidal component causes a maximum moment which can be computed with sufficient accuracy by the following equation in which p_o is the maximum amplitude of unbalanced pressure, L is one-half the wave length of the sine curve, and R is the radius of the shaft.

$$M = \frac{4}{3} p_o L^2/\pi^2 = p_o R^2/3 \qquad (5\text{-}6)$$

Both for dead load and for live load irregularities in loading may occur over the entire height because of variation in properties of the soil or for other reasons. These irregularities are likely to be intirely accidental in character. In order to account for them it is recommended that a standard irregularity corresponding to 10 percent of the lateral dead load design pressure be considered, with the variation in pressure considered to be that which corresponds to a sine curve distribution over a length of one-fourth of the circumference of the wall of the shaft. The moment for this single sine curve of loading is 1/3 greater than that for a beam having a length of one-fourth the circumference. This moment M is defined also by Eq. (5.6) with the amplitude of the unbalanced loading, in general being taken as 10 percent of the horizontal dead load pressure for the case. In computing the unbalanced pressure, the component of pressure loading due to water is neglected because this cannot be unsymmetrical.

If the wall of the silo is made of reinforced concrete, it is recommended that at least 0.25 percent of reinforcement be used in both faces of the wall, in both the vertical and horizontal directions. In most instances

this amount of steel will be sufficient to take care of the ovalling tendencies caused by the variation in properties of soil. Generally thicker sections and perhaps greater amounts of reinforcement will be required near the surface to withstand the loads developed there.

5.3.5 Vertical Force on Silo. Vertical loading is transmitted directly to the roof of the silo from the blast and additional vertical forces are transmitted by "negative skin friction" of the earth on the silo walls. In computing the latter force, account must be taken of the direction of relative motion of the silo and of the earth alongside. When an increased load is transmitted to the silo by friction from the earth, the vertical force in the adjacent earth is diminished. Consequently, the relative motions are changed, and the process of adding load to the silo is in a sense self-limiting. Furthermore, the maximum horizontal pressures on the silo may not occur simultaneously at all elevations, and consequently the frictional forces may not add fully. The behavior is a very complex one, and attempts to simplify the problem generally involve assumptions that are too conservative.

In general, the vertical force transmitted by friction on the silo walls can be computed from the magnitude of the shearing resistance of the soil adjacent to the silo and the lateral force. The shearing force transmitted to the silo wall cannot exceed the product of the coefficient of friction multiplied by the lateral force. However, the coefficient of friction used should be less than the tangent of the angle of internal friction of the soil, because the soil adjacent to the silo is generally disturbed by the construction operations. It is appropriate to take an angle of internal friction 5 degrees less than that which corresponds to the general mass of the material, in computing the shearing force transmitted to the wall of the silo. This shearing force should not, however, be taken as larger than the shearing resistance of the disturbed material near the silo if there is any other measure available of this shearing resistance. For cohesive materials the shearing resistance should not be taken as more than one-half the unconfined compressive strength of the material. It is suggested that the maximum vertical force in the silo wall and the maximum pressure on the foundations be computed on the basis of the following principles:

(1) Assume that reversal of direction of the shearing force occurs at about one-half the effective depth from the surface to the base. The effective depth is the net height minus the portion of the height with a sloping or wedge-shape profile near the top.

(2) The maximum vertical stress in the walls occurs at the point where the shearing force reverses in direction. This maximum stress is computed for the combined surface loading plus the shearing force of the upper part where the shearing forces are acting downward.

(3) The total load on the base of the silo is equal to the total load at the top plus the net force transmitted by shear. The net force transmitted by shear is zero in a homogeneous material, but it may not be zero if the base of the silo is founded on a firm or unyielding surface.

Where the material varies or has strata of different properties, the situation is much more complex. An approach to the problem may be made by assuming the direction of relative motion of the soil and the silo at various points along the height and by taking the shear in the direction to oppose this relative motion. In this procedure one computes a net force at the base of the silo, which is greater than that which corresponds to the load transmitted to the roof of the silo. One can compute a reduced force in the earth alongside the silo by assuming that the shear transmitted to the silo changes the vertical pressure in the soil uniformly over a distance corresponding to one silo diameter from the face of the silo wall. From the compressibility of the soil one can compute the deformations in the soil at various elevations and from the reduced pressures on the soil and the increased pressures on the silo one can compute the base displacements or estimate these. From these computations one can then recompute the relative motions and adjust the shears accordingly. This process is repeated until an agreement is reached between the assumed and derived values.

5.3.6 Dead Load Pressures on Tunnels. Consider a tunnel cross-section of the type shown in Fig. 5-4a or 5-4b, or a circular cylinder. In the latter case, let the designation B denote the diameter of the cylinder as well as the height. If the tunnel is driven by other than open cut methods, then the pressure p_v on the roof of the tunnel for granular material may be taken as given by the following equation:

$$p_v \leq w(0.50B + 0.58h - \bar{c}/w) \frac{1}{\tan \varphi} \qquad (5\text{-}7)$$

where h is the height of the tunnel, B the width of the tunnel, $\bar{c}$ the cohesive strength of the material, φ the angle of internal friction for the material, and w the weight per unit volume of the material. In general $\bar{c}$ may be taken as zero and the angle of internal friction adjusted accordingly. It can be seen from this equation that if

$$0.50B + 0.58h < \bar{c}/w ,$$

the pressure on the roof will be zero.

If the passageway is constructed by making an open cut, building the structure, and then backfilling, the pressure on the top will of course be equal to the weight of the material on top of the structure. It can even exceed this if the foundation of the passageway is firmer than that of the material alongside so that there is "negative" arching transferring more load to the roof. The horizontal pressures on the vertical walls of the box section shown in Fig. 5-4a can be computed as if the side walls were the same as a retaining wall of height h, loaded with an additional surcharge corresponding to the roof pressures computed above from Eq. (5-7). For the arched or circular tube sections, the pressures may be taken as radial and equal to the magnitude of p_v. An additional effect of ovalling for dead load is considered below.

5.3.7 Live Load Pressures on Tunnels. For the rectangular structure shown in Fig. 5-4a the live loading on the roof should in general be taken as

equal to the overpressure on the ground surface. If, however, the structure is buried with a cover at least half of the span, the maximum pressure on the roof may be reduced from the ground surface pressure. The reduction is given by the following equation:

$$\Delta p_v = \frac{H - 0.5B}{B} \sigma_s \qquad (5\text{-}8)$$

where σ_s is the "frictional part" of the shearing strength of the soil, which under the dynamic conditions may be taken as follows:

$$\sigma_s = 0.25 p_{so} \tan \varphi \qquad (5\text{-}9)$$

where p_{so} is the maximum surface overpressure and φ the angle of internal friction.

In no case should the reduction be taken as more than 1/2 the surface pressure; i.e., the minimum value of p_v is 1/2 the surface pressure.

The loading on an arch underground is described in Para. 5A.3. The loading on a cylinder may be computed as for an arch of 180-degree central angle. In general for a buried arch, if the average depth of cover over the arch is at least 0.25B, the tendency of the arch to buckle may be neglected, and the flexural component of loading due to the blast may be neglected. In such a case the pressure on the arch may be taken as uniform and equal in value to the surface pressure. If the average depth of cover is greater than 0.5B, the pressure may be reduced in the same way as for a flat-roofed structure, using Eq. (5-8) with H_{av} instead of H.

The live load pressures on the vertical walls of the box section may be taken as having the same ratio to the vertical pressure as is used for a vertical silo wall.

5.3.8 Virtual Mass of Soil Supported by Buried Structure. Very little information exists from which an estimate can be made of the mass of the soil which responds with a buried structure when it is loaded by shock. This virtual soil mass is analogous in some respects to the virtual mass of water used for an object submerged in water. However, the virtual mass of soil cannot be considered identical to the virtual mass of water because of the inherent shearing strength of soil.

Experimental evidence for rectangular structures buried to depths less than or equal to the span of the roof indicates the virtual mass may equal the total mass of the soil supported by the roof. Thus, for depths of burial less than or equal to the span of the roof for rectangular structures, it is recommended that all of the soil supported be assumed to respond with the roof. For greater depths of burial it is recommended that a rectangular block of soil defined by the plan dimensions of the roof with a depth not greater than the span of the roof be assumed. The data indicate this recommendation will probably provide a conservative result. Similarly a

conservative result is obtained if no soil is assumed to act with the walls of a rectangular structure.

The virtual mass for an arch or dome may be taken essentially the same as that specified for a rectangular structure. For the arch or dome the average depth of burial over the shell should be used, and the largest virtual mass should be limited to an average depth corresponding to one-half the span.

Virtual mass for the walls of the silo may be assumed equal to a thickness of soil not greater than the radius of the shaft.

5.3.9. Non-Uniform Pressure Around Tunnel. For shallow structures the pressures may be non-uniform from the dynamic loading. However, the non-uniformity is not important in case of the box section in which flexure is primarily the mode of action. For an arch or tubular section, the non-uniformity may be significant in that it introduces a different mode of behavior. This is considered in the design charts presented in APPENDICES 5A and 5B.

In addition to this kind of nonuniformity of loading, we must consider nonuniformity of loading arising from variation in properties of the material or from methods of construction. In general, the same degree of nonuniformity of dead loading as in the vertical silo should be considered, corresponding to a 10 percent variation in dead load pressure from one side to the other. In the case of the arch, the length over which this variation should be considered to take place is one-half the circumference from one footing to the other. In the case of the circular tube it should be one-fourth the circumference. No variation in vertical loading from live load need be considered, however, because this loading is not affected greatly by minor changes in the soil properties.

5.4 EARTH SHOCK AND SHOCK MOUNTING

5.4.1 Free-Field Air-Induced Earth Shock

a. Introductory Remarks. The ground motion resulting from a nuclear detonation is a complex combination of many effects, including air-induced shock, direct-transmitted ground shock, surface and reflected waves, coupled effects, and random motions. Because of the complex nature of the situation, it is convenient for design purposes to consider the earth shock resulting from a nuclear explosion as producing both systematic and random effects. Systematic effects can further be divided into two major types: (1) air-induced shock associated with the passage of an air shock wave over the surface of the ground, and the overpressure at the surface above the structure downward with such attenuation and dispersion as may be consistent with the physical conditions at the site; and (2) direct-transmitted ground shock arising from direct energy transfer from surface, near surface, or underground bursts. Random effects commonly include high frequency ground-transmitted shock, surface wave effects, reflections, refractions, etc. Which

particular effect is dominant and controls the design is dependent on such factors as weapon yield, point of detonation with respect to the ground surface, range from ground zero, depth of the structure, and geologic conditions.

At present it is possible to make reasonable estimates of the maximum values of displacement, velocity and acceleration that are associated with the air-induced shock effects, and in more restricted cases for the direct-transmitted ground shock effects, under more or less uniform geologic conditions.

Relationships for estimating air-induced effects, which often are the largest effects, are presented in the following. A more detailed treatment of the problem, including a discussion of layered media, and direct-transmitted effects is presented in APPENDIX 5C.

When structural systems or equipment are subjected to a base disturbance, as for example that arising from the ground motion associated with a nuclear blast, the response of the system is governed by the distribution and magnitudes of the masses and resistance elements. A knowledge of the response of systems subjected to such loadings is extremely important from the standpoint of design in order to proportion the structure so that it will not undergo complete collapse, and to protect the structure, equipment, and personnel from shock damage.

For purposes of assessing the relative effects in a structure, or the effects on secondary structures mounted on the soil within a primary structure, one of the simplest interpretations of ground motion data involves the concept of the response spectrum, which is a plot against frequency of the maximum response of a simple linear oscillator subjected to a given base input motion. Studies of the many shock spectra that have been determined from ground motion measurements, from both blast and earthquake sources, suggest that response spectra can be described in a relatively simple way in terms of the maximum values of displacement, velocity or acceleration. Concepts relating to both systematic and random disturbances are presented in this section and APPENDIX 5C. Brief comments on design to resist ground shock motions, and effects of structures on equipment shock response also are presented in APPENDIX 5C.

b. Notation. The notation used in this section is as follows:

- a = maximum vertical transient acceleration, in gravities
- c = seismic velocity of soil in vertical direction, in ft. per sec.
- d_e = maximum elastic component of vertical transient displacement, in in.; for a triangular pressure-time pulse $d_e = hp_{so}/2E$
- d_p = permanent vertical displacement after blast, in in.

E = Young's modulus of elasticity, in psi. For plane waves E is given by

$$E = \frac{(1 + \bar{\mu})(1 - 2\bar{\mu})}{(1 - \bar{\mu})} \rho c^2$$

where ρ is the mass density of the soil, $\bar{\mu}$ is Poisson's ratio, and c is the seismic velocity as defined above. For values of μ of 0.25 or less, the relationship is approximately $E = \rho c^2$, and for soil with a density of about 115 lb. per cu. ft. an approximate value of E is

$$E = 25{,}000 \text{ psi} \left[\frac{c}{1000 \text{ fps}}\right]^2$$

h = depth to which shock extends in time t_1, in ft.;

$$h = ct_1 = 400 \text{ ft.} \left[\frac{c}{1000 \text{ fps}}\right] \left[\frac{100 \text{ psi}}{p_{so}}\right]^{0.6} \left[\frac{W}{1MT}\right]^{1/3}$$

L = quantity in units of ft., a function of overpressure and duration, used in pressure and velocity attenuation relationship

p_{so} = peak overpressure in shock wave, in psi

t_1 = effective duration of shock, corresponding to a triangular pressure pulse having the same impulse as the actual shock, in sec., (see Ref. 15);

$$t_1 = 0.40 \text{ sec} \left[\frac{100 \text{ psi}}{p_{so}}\right]^{0.6} \left[\frac{W}{1MT}\right]^{1/3}$$

t_r = effective velocity pulse rise time, in sec.; field observations indicate that

$$t_r \simeq \frac{1}{2} \frac{y}{c}$$

for a homogeneous medium

v = maximum vertical transient velocity, in ft. per sec.

W = yield of weapon, in Megatons

y = depth below surface to point considered, in ft.

α = attenuation factor for velocity or stress

Subscripts: "s" denotes the surface and "y" denotes a distance y below the surface

c. Free-Field Air-Induced Earth Motions at Surface

Maximum Transient Vertical Displacement at Surface

The elastic component of the maximum transient vertical displacement in homogeneous material may be taken as follows:

$$d_{se} = 10 \text{ in.} \left[\frac{p_{so}}{100 \text{ psi}}\right]^{0.4} \left[\frac{1000 \text{ fps}}{c}\right] \left[\frac{W}{1MT}\right]^{1/3} \qquad (5\text{-}10)$$

The permanent vertical displacement depends on the overpressure and on the plastic properties of the soil in the upper 50 to 100 ft. It is often of negligible magnitude for overpressures less than 100 psi, but for soft and weak soils it can be as much as 5 or 6 in. at the surface, even at an overpressure as low as 100 psi. If static stress-strain curves for the soil are not available from which to estimate the permanent displacement, it is suggested that it be taken as follows:

$$d_{sp} = \frac{p_{so} - 40}{30} \text{ in.} \left[\frac{1000 \text{ fps}}{c}\right]^{2} \qquad (5\text{-}11)$$

In this equation, c is the seismic velocity near the surface. When the equation is used, a cut-off in permanent displacement occurs at 40 psi. Available evidence indicates that permanent displacements generally are of a negligible magnitude at pressures below 40 psi; accordingly it is recommended generally that d_{sp} be taken as zero for pressures less than 40 psi. In exceptional cases there may be reason to estimate the permanent displacement for lower pressures from known stress-strain properties when the soil properties are available.

The maximum transient elastic vertical displacement in a layered or in a non-homogeneous system can be different from Eq. (5-10). For a rigid layer near the surface, but at a depth greater than h, there can be a complete reflection which at most could double the value of d_{se} arising from the near surface strains. For a system with variable properties, or layers, the value should be computed for several positions of the shock, taking account of the values of c for each layer, and adding up the instantaneous values of strain so determined.

Maximum Transient Vertical Velocity at Surface

The maximum transient vertical velocity can be taken as:

$$v_s = c p_{so}/E$$

whence

$$v_s = 4.0 \text{ fps} \left[\frac{p_{so}}{100 \text{ psi}}\right] \left[\frac{1000 \text{ fps}}{c}\right] \qquad (5\text{-}12)$$

Maximum Transient Vertical Acceleration

This is computed by assuming a rise time for the maximum velocity (or maximum pressure) of about 0.001 sec., from which it follows that

$$a_s = 150 \text{ g} \left[\frac{p_{so}}{100 \text{ psi}}\right] \left[\frac{1000 \text{ fps}}{c}\right] \qquad (5\text{-}13)$$

In the last two equations, one must use the surface seismic velocity. However, the maximum acceleration is not necessarily related to the maximum velocity, but may be larger than the value computed from Eq. (5-13). Therefore it is recommended that even for high seismic velocities, a value of c no greater than 2000 ft. per sec. be used. In accordance with the discussion given in SECTION 5C.7 and Table 5C-1, Eq. (5-13) has taken into account a factor of 2 which has been introduced to account for normally expected levels of damping and complex oscillations. Other situations may be handled with the use of Table 5C.1.

Free-Field Horizontal Effects at Surface. For horizontal surface effects, take the maximum displacement as 1/3 the vertical, the maximum velocity as 2/3 the vertical, and the maximum acceleration equal to the vertical.

d. Free-Field Effects at Depth. The displacement, velocity, and acceleration are attenuated with depth. Although experimental data are scarce, the following basis seems reasonable for computing the effects at a depth y.

Vertical Displacement at Depth y

The difference in displacement between the surface and the depth y cannot exceed the sum of the maximum strains between these points, and can be considerably less than this. Between the surface and a depth of 100 ft., the maximum possible elastic strain, assuming no attenuation of pressure, gives an upper limit to the elastic component of the differential displacement, of magnitude

$$4.8 \text{ in.} \left[\frac{p_{so}}{100 \text{ psi}}\right]\left[\frac{1000 \text{ fps}}{c}\right]^2 \qquad (5\text{-}14)$$

The actual difference in deflection may be taken as one-half this value, which is considered to be a more reasonable value and considered to vary linearly with depth down to 100 ft. The permanent vertical displacement attenuates rapidly, and can be assumed to vary linearly from the surface value, given by Eq. (5-11), to zero at a depth of 100 ft.

The change in total maximum vertical displacement with depth, for homogeneous material, may be taken as indicated in Table 5-1. This table indicates no change in maximum deflection below a depth of 100 ft.

Vertical Velocity at Depth y

The vertical velocity at depth y is attenuated roughly in the same way as the maximum stress, or

$$v_y = \alpha\, v_s \qquad (5\text{-}15)$$

where

$$\alpha = \frac{1}{1 + y/L}$$

and

$$L = 300 \text{ ft.} \left[\frac{100 \text{ psi}}{p_{so}}\right]^{0.6} \left[\frac{W}{1MT}\right]^{1/3} \qquad \text{for } p_{so} \leq 500 \text{ psi}$$

$$L = 138 \text{ ft.} \left[\frac{100 \text{ psi}}{p_{so}}\right]^{0.1} \left[\frac{W}{1MT}\right]^{1/3} \qquad \text{for } p_{so} \geq 500 \text{ psi}$$

Vertical Acceleration at Depth y

The time of rise of the maximum velocity from an initial zero value of velocity can be taken as one-half the transit time of the shock wave from the surface to the depth considered. However, the maximum acceleration can be considered to be twice the value obtained from the assumption that the maximum velocity is obtained linearly. This leads to the relation:

$$a_y = 2g \frac{v_y}{t_r} \frac{1}{32 \text{ ft/sec}^2} \tag{5-16}$$

The rise time of the peak velocity should not be taken as less than 0.001 sec. This procedure gives less attenuation of acceleration in rock than in soft soil, which is reasonable. If no attenuation of velocity or pressure with depth is assumed, the use of Eq. (5-12) and (5-16) give the following result:

$$a_y = 5g \left[\frac{p_{so}}{100 \text{ psi}}\right] \left[\frac{100 \text{ ft.}}{y}\right] \tag{5-17}$$

Horizontal Motions at Depth y

The ratios of peak horizontal to peak vertical displacements, velocities, and accelerations at depth y are to be taken as 1/3, 2/3, and 1, respectively.

e. Illustrative Example. As an example of the use of the relations given herein, consider the case of an 8 MT weapon for an overpressure of 200 psi, and a homogeneous soil having a seismic velocity of 2000 ft. per sec. (corresponding to a sandy-silt). The estimated values of vertical and horizontal maximum displacement, velocity, and acceleration at depths of 0, 50, and 100 ft. are given in Table 5-2.

It is readily apparent from the table that the surface deflection comes mainly from deformations at fairly great depths, up to 1000 ft. or so. Consequently, if the lower levels of the subsoil are substantially harder, the

deflections will be decreased. However, the difference in deflection between the surface and 50 ft. or 100 ft. will be virtually unaffected.

f. Design Shock Spectra. For either the relative effects in a structure, or the effects on equipment within a structure placed in the soil, the simplest interpretation of the earth motion data involves the concept of the response spectrum, which is a plot against frequency of the maximum response of a simple linear oscillator subjected to a given input motion. From the many response spectra that have been determined for ground shock motions, the response spectra can be described in a relatively simple way in terms of only the maximum values of the ground particle displacement, velocity, or acceleration. To do so we make use of a logarithmic plot of maximum velocity (or really the circular frequency times the displacement called the pseudo-velocity) versus frequency, as in Fig. 5-5. Diagonal lines drawn on the plots represent constant values of displacement or acceleration, and consequently one can read from the one plot values of the acceleration, pseudo-velocity, or displacement response spectra for a system having a particular frequency. The shock spectrum shown in Fig. 5-5 is typical of such spectra for earth shock. It is noted that the spectrum consists of three straight lines which are actually bounds to the actual spectrum, determined as follows. For a more complete discussion of the theory upon which the spectrum concepts are based see Ref. 15.

For the particular point and direction of motion considered determine from the preceding discussion the maximum values of displacement, velocity, and acceleration. Then draw the spectrum bound for the system by the use of three straight lines:

A. A line parallel to the lines of constant displacement, drawn with a magnitude equal to the maximum displacement.

B. A line of constant velocity drawn with a magnitude of 1.5 times the maximum velocity.

C. A line parallel to the lines of constant acceleration, drawn with a magnitude equal to the maximum acceleration.

The heavy spectrum lines in Fig. 5-5 are consistent with a typical set of conditions at or near the surface for a bomb in the MT range, with a soil having an acoustic velocity of about 2000 ft. per sec., with an over-pressure of 200 psi, and a yield of about 8 MT. Also shown is the spectrum bound corresponding to the 100 ft. depth. The values of displacement, velocity and acceleration used in sketching these bounds correspond to the computations described in SECTION 5.4.1e and listed in Table 5.2. From the plot it can be seen, for example, that the maximum response of a piece of equipment having a frequency of 23 cycles per second would be 0.50 in., with a maximum acceleration of 25 g at or near the surface.

For a discussion of recommended bounds in cases involving trans-seismic and subseismic conditions refer to SECTION 5C.4, and for cases involving combined random and systematic pulses, refer to SECTION 5C.7.

5.4.2 Shock Mounting. This section is concerned with the problem of attachment of equipment (mechanical, electrical, hydraulic, etc.) to the protective structure. The equipment must remain attached throughout a blast and must function in the post-blast state. It is obvious that the attachments must have sufficient strength to transmit the forces which are associated with the equipment accelerations and with the relative distortions of structure and equipment. The stiffness of attachments must be considered not only in relation to its influence on the magnitudes of transmitted forces but also from the point of view of possible limits of acceptable relative displacements of equipment and structure.

Since the problem relates to the mounting of equipment, rather than to the articulation of major structural components, it can be assumed that the attached mass is relatively small in comparison with the mass of the structure. It follows that the attachment forces are negligible in comparison with the direct effects of the blast, and the motion of the structure is nearly independent of the forces transmitted through the attachments. Motion of the structure is taken as the basic input for which the mounting must be designed. These input data must be obtained from an analysis of the response of the structure to ground shock and air blast, or must be assumed.

Maximum accelerations or displacements which can be tolerated by the equipment must be known or computed. For complex items, such as electronic equipment, this information should be supplied by the manufacturer. The permissible accelerations and distortions of many other items, such as piping, ductwork, machinery bases, etc., often can be investigated directly by the mounting designer.

A much more detailed discussion of design to resist ground shock motions is given in SECTION 5C.8.

5.4.3 Provision for Relative Distortion of Equipment and Structure. When equipment must be connected to the structure at two or more points, and when significant relative displacements of these points are anticipated, the capacity of the equipment and attachments to accommodate such displacements must be investigated. Cases of this kind are not limited to the obvious situation in which the equipment is attached to two structures having independent motion components. Quite often structures are designed to undergo substantial distortion, particularly in flexural modes. A few examples are shown in Fig. 5-6; in each example, points a and b undergo significant relative displacements. This displacement may be either elastic or elastic-plastic. If some plastic distortion is anticipated its magnitude may be very sensitive to small changes in the assumed loading on the structure. If this is the case relative displacements should be computed on the assumption of maximum structural distortion; i.e., distortion corresponding to conditions when the structure is at the point of collapse.

It should be emphasized that relative displacement of attachment points may be accommodated by elastic or elastic-plastic distortion of the equipment, by flexible joints, slip-couplings or other devices incorporated in the equipment, by elastic or elastic-plastic distortion of the attachment,

or by some combination of these factors. It may be quite unrealistic to attempt to supply all of the required accommodation in the attachments. In the case of piping or conduit, for example, provision of bends or loops rather than a straight run between the connected points may permit the entire relative motion to be absorbed by flexural distortion of the pipe.

5.4.4 Nature of Elastic Systems Comprised of Mounted Equipment. In general any piece of mounted equipment comprises a multi-degree-of-freedom elastic system (or elasto-plastic system) which responds to the motion of its support points (points of attachment to the structure). If the equipment is so connected to the structure that relative distortions of the structure can be accommodated without serious stresses in equipment and attachments, a desirable condition, the stresses in the equipment and forces transmitted through the attachments will be primarily a function of accelerations of the equipment. The major problem of analysis thus is the determination of equipment accelerations. The products of equipment masses (concentrated or distributed) and corresponding accelerations represent a loading for which the corresponding stresses and support forces can be found by conventional methods of stress analysis.

Every system has many degrees of freedom and corresponding modes of motion, and the total motion is comprised of the sum of the responses in each mode. Fortunately most systems have only a very few, easily recognized modes of predominant significance which contribute most of the response to a specified direction of support motion. Consequently, it usually is sufficient to determine the response in each (often only one) of these predominant modes. When it is deemed necessary to determine the response in more than one mode, advantage should be taken of the fact that peak values of stresses and reactions in the separate modes are unlikely to occur simultaneously. Thus the combination of values from the separate modes should be based on probability consideration.

In some instances the flexibility of a piece of equipment and its attachments may be limited almost entirely to the latter. This would, for example, be the case if an electric motor were attached to the structure by relatively soft spring mountings. In other cases the attachments may be very rigid and the equipment may be relatively flexible. An example of the latter would be piping having a relatively small ratio of diameter to distance between points of support.

In many instances for which the equipment has a mass distributed over considerable length, or area, it is convenient to approximate the distributed mass by one (or a few) mass concentrations.

5.4.5 Design of Mounted Equipment to Resist Shock. In a typical case, an underground structure may be considered to move with the ground in accordance with the free-field motions at or near the base of the structure. Consider a situation where the motions are such as to lead to the response spectrum for design shown in Fig. 5-5. If a piece of equipment is to be mounted in the structure, the equipment must be designed for the motions it would receive. This response is determined by the frequency of the system

composed of the piece of equipment, its mounting bracket or connections, and the part of the structure to which it is attached. In general the structure is rigid enough so that all parts of the structure have the same motions and consequently the input motion for which the equipment is to be designed is the free-field earth motion.

If the equipment is a heavy, compact element mounted on a bracket, one must make an estimate of the natural frequency of the system. It will be possible in most cases to assume that the point of attachment of the bracket to the wall of the structure is a fixed point of support. Then from the flexibility of the bracket and the magnitude of the supported mass, one can compute the natural frequency. This can be estimated fairly well by determining what the deflection of the system would be in the direction of motion due to a force equal to the weight of the supported element. If this deflection is x_s, then the frequency f is approximately:

$$f = \frac{1}{2\pi}\sqrt{\frac{g}{x_s}} \tag{5-18}$$

where g is the acceleration of gravity.

For example, consider a piece of equipment which with its attachment plates and bolts weighs 1000 lb., bolted to a plate which is welded to the flanges of two channels, and attached to the wall of a structure as shown in Fig. 5-7.

The channels have a web thickness of 0.51 in. and a net height of web of 11.0 in. The spring constant for the two channels, each 1 ft. long, considered as deflecting without end rotation, because of the fixity of the web by the flanges, may be computed. The deflection due to a weight of 1000 lb. is found to be 0.014 in., and by use of Eq. (5-18), one obtains

$$f = 26.5 \text{ cycles per sec.}$$

The maximum static stress in the web from flexure is 5300 psi.

For the input data given, it is found from Fig. 5-5 that the acceleration response at a frequency of 26.5 cycles is about 30 g. This means that the equipment mass will be subjected to a maximum acceleration of 30 g and it also means that the bracket will have a stress of 30 times the stress computed for the weight of the equipment, or 159,000 psi, in addition to the static stress, or a total of about 164,000 psi. The bracket is clearly overstressed.

It is not necessarily true that strengthening the bracket will work with full effectiveness in reducing the stress, because adding to the strength at the same time adds to the stiffness and attracts more force because of the consequent increased accelerations response. For example, doubling the number of channel supports increases the frequency to 37.6 cycles per second, and gives an acceleration response of 44g, which results in a stress, including static stress, of 125,000 psi.

On the other hand, if the bracket were subjected to an input motion only one-fourth as great, the spectrum response values would be decreased to one-fourth their value from Fig. 5-5, and the net stress would correspond to an acceleration of about 7.5g, or 40,000 psi, plus the static stress of 5300 psi, which probably would be acceptable.

For more detailed discussion of shock mounting of various types of equipment, piping, etc., reference should be made to Refs. 6 and 15, which give examples of several types of mounting. Also, additional discussion of this problem is given in APPENDIX 5C, particularly SECTION 5C.8.

In general, it is desirable to provide as much flexibility in the mounting as possible without sacrificing strength, in order to keep the response as low as possible, both for the equipment and the mounting itself.

5.4.6 Design Stresses in Shock Mountings. If the forces transmitted through the attachments are determined on the basis of elastic behavior it should be safe to proportion the attachments for yield stresses at peak transmitted forces. If brittle materials are avoided the plastic distortion available generally will be substantially larger than the elastic distortion which occurs up to the point of yield. Consequently, actual fracture is not likely.

It is not feasible to recommend general stress levels for use in the equipment itself since these depend on the function of the equipment and the extent to which that function would be impaired by large strains. For those items involving ductile materials and where plastic strains would not impair the post-blast function, yield values of stress will be acceptable.

5.5 NUCLEAR RADIATION

5.5.1 General. The levels of nuclear radiation for which an installation is to be designed will be determined by means of a target analysis as discussed in SECTION 2. Significant types of radiation for protective construction are initial and residual gamma radiation and neutron radiation. The intensities of these types of radiation at any point in space are functions of weapon yield, distance, meteorological conditions, time and, for neutron radiation, weapon design.

Tolerable accumulated doses for personnel are discussed in SECTION 4 along with the vulnerability of certain types of equipment to nuclear radiation. Normally only electronic equipment is vulnerable to nuclear radiation and that only to neutrons. However, the range of vulnerability of that equipment may be great depending on the arrangement of the circuitry and the type of components used (transistors are especially vulnerable).

5.5.2 Shielding Requirements. Given the input radiation levels in initial gamma, fallout, and neutron, and the acceptable levels for personnel or equipment, the required transmission factor or its reciprocal, the reduction or attenuation factor, can be readily determined.

5.5.3 Shielding Effectiveness, Gamma Radiation. The effectiveness of various materials for shielding against initial and residual gamma radiations may be determined from Figs. 5-8 and 5-9. These dose transmission factors, however, are for a broad beam of radiation impinging on a thick plane shield with very large dimensions in the plane of the shield and with no nearby ceiling, floor or walls from which radiation may scatter. Procedures for determining the shielding from fallout or residual radiation afforded by above and belowground structures are available in Ref. 7. The pertinent figures of Ref. 7 are included in APPENDIX 5E together with some tabular forms indicating how the material is used in a calculation of the shielding effectiveness against fallout radiation of a given structural configuration.

5.5.4 Shielding Effectiveness, Neutron Radiation. Neutron attenuation is a more complex phenomenon than that of gamma attenuation, since several phenomena are involved in the former. First, the very fast neutrons must be slowed down to the moderately fast range. Then the moderately fast neutrons have to be decelerated into the slow range by means of an element of low atomic weight. Water is very satisfactory for this purpose because its two constituent elements, i.e., hydrogen and oxygen, both have low atomic weights. The slow (thermal) neutrons must then be absorbed. This is not a difficult matter since the hydrogen in water will serve the purpose. Unfortunately, however, most neutron capture reactions are accompanied by the emission of gamma rays. Consequently, sufficient gamma attenuating material must be included to minimize the escape of capture gamma rays from the shield.

5.6 THERMAL RADIATION

5.6.1 General. Thermal radiation intensities from large yield weapons are of significant levels for the ranges at which protection of military installations may be considered. For example, at 3000 ft. from a 1 MT surface burst weapon where a peak blast pressure level of about 160 psi would exist, the total thermal energy delivered under excellent visibility conditions would be about 1000 cal/cm^2. This intensity indicates that noncombustible materials must be used for hardened construction, and further that precautions must be taken to avoid significant loss of structural materials through surface erosion from the high thermal input.

In addition to the intensive thermal radiations at potentially interesting distances from weapon detonation points, the air temperatures will rise considerably during the passage of the shock front.

5.6.2. Radiation Intensities. Curves of radiant energy vs. alert range (Vulnerability Radius) for various weapon yields have been presented in Fig. 2-12.

5.6.3 Air Temperatures. Estimates of maximum air temperatures in the blast wave have been made in Ref. 17. For overpressures of 40, 100, 200 and 400 psi, the maximum air temperatures are given therein as about 400°K, 1100°K, 4000°K, and 15,000°K. The significance of these temperatures has not been established.

5.6.4 Effect of Thermal Radiation. For the very high radiative inputs at the close-in ranges, loss of material through spalling can be expected. Theoretical estimates have been made (see Refs. 8 and 9) of this effect but the results are not very reliable. The limited field test data that have been obtained indicate that the erosion is probably considerably less than would be estimated from the melting effects and temperature distributions that have been computed.

Temperature profiles for steel and concrete have been estimated theoretically, (Ref. 8) for 1, 10 and 20 MT weapons at the 100 psi range. It is apparent that high temperatures are confined to close distances from the surface.

It is concluded that, at ranges corresponding to blast pressures of 100 psi or greater, metal parts should not be exposed unless erosion of surface material is not critical. In general, the erosion would not be significant as far as structural strength is concerned but it could affect the operation of rollers, etc. Spalling of concrete may also be expected at these ranges. Two inches of concrete should provide sufficient insulation for metal parts at the 100 psi range.

5.7 UTILITY SYSTEMS*

5.7.1 General. This section deals with the selection and design of the utility systems and building services which are deemed necessary to insure a full operational capability prior to, during, and following an attack. It is intended to bring into focus the technical and economical feasibility of using available mechanical and electrical components for the various categories of hardened facilities under consideration, and also to review the more recently developed sophisticated methods for generating electrical power and their potential use in connection with protective construction. The primary utility systems and building services covered in this section are as follows:

a. Electrical power
b. Heat sinks
c. Air supply and conditioning
d. Water supply
e. Fire protection

5.7.1.1 Operational Condition. The studies and investigations required to establish a basis for design of the utilities and services revolves around the operational condition of the facility prior to, during and following an attack. The influence of the operational condition on the utilities should be evaluated in light of the following factors:

a. Degree of Activity. The approach to a system design invariably will be steered in accordance with the degree of activity to be experienced prior to and following an attack. Economy of operation, as a

* Prepared by Mr. S. S. Greenfield, Chief Mechanical Engineer, Parsons, Brinkerhoff, Quade and Douglas, New York.

design parameter, may override compactness and simplicity in the case of a facility which must be fully operational at all times. A different posture may well be taken in regard to the utilities design for the facility which will be exposed to prolonged standby periods with only a full operational condition following an attack.

b. Button-up Period. The length of the button-up period and the requirement for a restored capability if any, will obviously affect the type of power source, water supply, heat sink and the storage of consumables.

c. Environmental Conditions. Following an attack, the utilities and services must be capable of maintaining the proper environmental conditions to insure a high level of personnel efficiency and electronic equipment operation without relying on outside support.

The importance of establishing the operational condition of a facility for design purposes cannot be overemphasized. It is incumbent upon the Using Agency in conjunction with the design group to formulate such conditions at the time feasibility studies are being made. Some of the more probable operational conditions to be encountered are presented in a Corps of Engineers Design Manual (Ref. 5.7-1).

5.7.1.2 Level of Protection. The hardness of a facility, in terms of overpressure resistance and biological shielding, as it affects the configuration and depth of cover will influence the utilities design.

5.7.1.3 Geographic Location. Another significant factor affecting system design is the climate and geologic formation that will surround a proposed installation which in turn governs the conditions of outside air available for ventilation and disposal of heat rejected from power generation and air conditioning equipment. The prevalence of underground water and the initial earth or rock temperature are also dependent on geographic location.

5.7.1.4 Communications Requirements. The maintenance of communications between military facilities during emergency periods is mandatory. This requires the assurance of a reliable source of power, generally closely regulated in voltage, frequency and wave form.

5.7.1.5 Shock Transmission. Consideration must be given to ground shock in the design of the utilities and services to insure the functional survival of the essential components. The degree of hardness to be designed into a specific installation may require solutions which might vary from the simple application of standard commercial vibration isolators to the use of extremely complex packages of supporting devices.

5.7.2 Electrical Power. A major consideration in the design of a hardened installation is the selection of the electric power system which would be best suited to meet operational requirements. The power requirements may range over a spectrum of kilowatts to megawatts.

5.7.2.1 Basic Considerations. Any investigation of the applicability of various power sources for hardened military installations should take into account the following important basic considerations:

a. Power Demand. The utilization of electric power falls into two main categories. One encompasses the special equipment or "hardware," namely, communications, data processing and launch facilities. The other covers the support utilities and services. The Design Engineer must look to the Using Agency to establish the magnitude and characteristics of the electrical power demand for the "hardware" which governs the determination of the power demand for the support services. The latter is the responsibility of the Design Engineer.

Since redundancy is commonly employed to insure the operational reliability of the "hardware" it is vitally important, on the part of the Using Agency, to apply demand factors to the connected loads. Past experience has indicated that this factor has not been given proper consideration with the result that power demands have been estimated considerably higher than actually necessary.

b. Voltage and Frequency Regulation. The ultimate utilization of the electrical power, particularly for electronic equipment, may require close voltage and frequency regulation. Indeed, such criteria may dictate the need for special regulatory controls and the use of non-standard alternators and exciters.

c. Service Interruptions. It is extremely important to establish critical loads, that is, equipment and service which cannot tolerate an interruption of power supply. Experience has shown that electronic equipment, particularly that using vacuum tubes, may suffer damage or malfunction in the event of a relatively short time interruption of power supply.

d. Commercial Power. In the consideration of possible power sources for any specific facility, particularly if located in the United States, the availability of the vast interconnected commercial public utility network is strikingly evident. However, peculiar to any commercial power source is the extensive system of overhead distribution lines which are susceptible to faults caused by storms or sabotage. In some instances the character of the power capable of delivery to the facility does not meet the stringent voltage and frequency requirements which may be imposed by special electronic equipment. While it is true that there may be long periods of stand-by or alert operations during which consideration could be given to the use of commercial power, it must be recognized that following an attack a hardened facility must be self-sufficient and consequently a protected power source of sufficient capacity to insure a power supply for the critical loads must be available. If a significant portion of the electrical power demand is required for the operation of critical electronic and building service loads, then the use of commercial power becomes less attractive. Several studies have been made for specific installations in which critical loads were predominant. These studies have indicated that it is far more satisfactory and incidentally, also more

economical, to provide a generating plant within the installation itself to serve all the load and eliminate any connection to a commercial power source.

5.7.2.2 Power Generating Plant. The selection and design of a power generating plant to be installed in the protected area of the facility should take into consideration a number of factors which result from the fact that the facility must operate in a button-up status. These factors are:

a. Heat Rejection. The problem of dissipating rejected heat resulting from the thermo-dynamic cycle and mechanical inefficiencies has been a persistently difficult one to overcome. This is particularly true for the button-up period. The rejected heat and the manner in which it manifests itself has a significant effect on the cost of the over-all utility system by virtue of its impact on the heat sink. It may be that power sources which are initially more costly can be justified on the basis of overriding costs of providing a heat disposal system. In this connection consideration should be given to the possibility that the rejected heat can be put to useful purposes.

b. Geographic Elevation. The geographic location of the installation will determine its elevation above sea level, and this might significantly affect the performance of power sources using a fuel-air combustion cycle. For example, a two-cycle diesel engine or an open-cycle gas turbine must be de-rated according to its elevation above sea level.

c. Vulnerability. It is apparent that a power source employing a fuel-air combustion cycle must communicate with the outside atmosphere for air intake and exhaust. Connections to the surface generally involve a large expenditure, particularly in the case of deep underground facilities and they introduce a potential weak spot in the integrity of an otherwise well-protected facility. This consideration warrants a thorough investigation of the use of power sources independent of outside air for combustion.

d. Ruggedness. The apparent tendency toward the protection of facilities against the detonation of extremely high yield weapons brings into focus the ability of power generating equipment to withstand vibrations resulting from air-induced or ground-transmitted shock. It may be that shock isolating supports will have to be used.

e. Compactness. High power to volume ratios are especially desirable for cut and cover installations where the construction of hard space involves large expenditures. On the other hand, in deep underground facilities the cost of excavating additional chamber space for power generation components is not nearly as critical.

f. Fuel Supply. A major problem for consideration in connection with power generating equipment is the supply and storage of fuel, particularly for use during the button-up period. The ability to utilize different types of fuel, for instance, a diesel engine which can be switched from gas to fuel oil and vice versa, may present decided advantages. It is

evident that a power source which would not require continuous fueling has decided advantages in this respect, on the other hand, such a power source generally involves high initial cost.

g. <u>Operation and Maintenance</u>. It is evident that power sources which are less complex and require fewer auxiliaries with a correspondingly moderate complement of spare parts, can more readily be maintained with normally competent personnel. In instances where the facility is located in a remote area, this could be a decided advantage. Obviously, a power source which is fully automatic and therefore can operate unattended, is highly desirable. However, this is not generally feasible, and a plant that can be operated by personnel not highly skilled, is usually an acceptable compromise.

h. <u>Initial and Operating Costs</u>. While it is true that these factors are of importance in the final determination of any power source, power plants that may not be economically competitive in themselves in some instances can be justified on the basis of overriding operational advantages. It is essential that these factors be closely weighed in the selection and design of the power generating plant.

5.7.2.3 <u>Alternative Power Generation Equipment</u>. Consideration of the basic requirements discussed above should lead to an investigation of the various types of power generating equipment which are now available. These include the use of conventional prime movers, such as diesel engine and gas turbine combined with alternators and more recently developed power sources, such as nuclear reactors and fuel cells.

a. <u>Diesel Engine Plant</u>. One of the most common and versatile types of power sources employs a diesel engine prime mover driving an alternator. Standard commercial diesel generator units are available in a variety of sizes to the extent that singly or in multiples they could satisfy the requirements of practically all categories of hardened facilities. Although the thermal efficiency is subject to Carnot-cycle limitations, efficiencies on the order of 30% to 35% can be expected even in the smaller size plants. The engines can operate equally well using either liquid or gaseous fuels or a combination of both. Surface connections are required for combustion air intake and the exhaust of combustion gases. Fortunately a significant percentage of the cycle heat can be ejected along with the exhaust gases. However, about one-third of the heat input must be dissipated to a heat sink. If ebullient cooling of the engine jackets is adopted, rejected heat in the form of low pressure steam can be vented to the atmosphere which would significantly reduce the load imposed on the heat sink. A study should be made to determine the economic feasibility of recovering some of the rejected heat either in the form of hot water or steam for heating and absorption refrigeration. Consideration also should be given to the utilization of superchargers to correct de-rating for high geographical locations above sea level. Table 5-3 presents some of the salient characteristics of a medium size diesel electric plant.

b. Open-Cycle Gas Turbine Plant. For this type of power generating equipment, two types of cycles appear to be the most applicable, namely, a simple cycle with zero regeneration and an 80% regenerative cycle. A third type of cycle utilizing a combination gas turbine-steam turbine cycle, although somewhat more efficient than the 80% regenerative cycle, is considerably more complex, which generally overrides the advantage of savings in fuel. Even with the limited range of sizes available, gas turbines can be considered adaptable to the power requirements of the majority of installations under consideration.

Cycle efficiencies of gas turbines range from 15% to 27%. The units can operate equally well on either liquid or gaseous fuel. One extremely desirable feature of the turbine cycle is that the heat rejection problem is less severe in that almost all the cycle heat, except for useful work, is ejected along with the exhaust gases. This advantage becomes less evident if heat recovery devices are found to have practicable application for heating and absorption refrigeration. The principal disadvantage of the turbine cycle is the enormous demand for air, with resulting large expenditures for surface connections and blast closures. Air induction losses and exhaust backpressures resulting from the dynamic losses of air passing through long shafts or tunnels and across blast closures may significantly de-rate the turbine. Booster fans could be used to minimize these effects, but of course, they require additional power for operation. De-rating due to altitude is an inherent disadvantage of the turbine cycle and cannot be corrected by the use of supercharges as in the case for diesel engines. While turbines possess a greater power to weight and power to volume ratio than diesel engines, this factor must be evaluated against the other disadvantages. (Refer to Table 5-3 for comparative characteristics.)

c. Nuclear Power Plants. The requirement for atmospheric air to support the combustion cycle for fossil fuel fired power plants will generally introduce large expenditures for surface connections and blast closures. In fact, when considering protective construction to resist blast effects from surface overpressures in excess of 1000 psi, the design of large closures may present problems beyond the present "state of the art". For facilities which require resistance to such high overpressures, and recognizing that with chemical air revivification, personnel could subsist for extended button-up periods without outside air (Ref. 5.7-2), the generation of power, particularly for large facilities, by other than air-aspirating devices, warrants investigation. It appears therefore, that thermal energy produced by nuclear fission should be given consideration.

There are many reactor types which use nuclear source material and some have promise of civilian and military application. Major classifications of reactors under extensive study at the present time include water-cooled, gas-cooled, organic and liquid metal cooled reactors. While the technical feasibility of power production for a number of different types of reactors has been demonstrated, the cost of generating the power still remains relatively high. In addition, the increased requirement for heat absorption

presents a problem, the solution of which might be rather costly. However, as a potential power source for hardened facilities where protection is paramount, nuclear energy has the unique characteristic of nondependence on fuel and combustion air and perhaps the over-all protection may override the higher capital and operating costs.

The nuclear power projects presently under study and construction by the Military, are adequately covered in two papers (Ref. 5.7-3 and 5.7-4). It is noted that pressurized water reactors predominate, followed by gas-cooled reactors. The pressurized water reactor is representative of the type that generates steam for use with a conventional turbine generator unit. For this type of equipment, the plant is considerably more complex than either the conventional diesel engine or gas turbine plants previously discussed. The gas-cooled reactor is used in conjunction with a closed-cycle nitrogen or helium gas turbine. It can be made quite simple but such simplicity sacrifices thermal efficiency. The over-all energy conversion for both types is limited by Carnot-cycle efficiency.

The design should contemplate an underground heat sink developed within the protected area. In the case of an extended button-up period, the magnitude of the problem becomes apparent in that 75% to 80% of the thermal power of the reactor manifests itself as rejected heat. Operating procedures and the employment of certain auxiliaries basically altering the temperature level of the heat rejection process could minimize heat sink requirements but at some sacrifice of plant efficiency.

As an illustration of this problem an analysis of the heat balance of an applicable nuclear-steam power plant, the PM-1 (Ref. 5.7-5) producing one megawatt of net electric power follows. (Refer to Fig. 5-10 for cycle diagram.)

During operation prior to an attack, the reactor-turbine plant might operate with initial steam conditions of 280 PSIA, an exhaust of 6" of Hg absolute, a heat source of 7.3 megawatts (reactor power), and a heat rejection of 5.8 megawatts. The heat sink would be outside air and the heat rejection equipment would consist of a standard tubular surface condenser and cooling towers or spray ponds located near the protected facility at grade level. The blast effects of a weapon detonation would therefore render them functionally inoperable, requiring an underground water storage reservoir to serve as a hardened heat sink during the attack and post-attack periods.

With water stored initially at 40°F and allowed to reach a maximum temperature of 160°F, the heat rejected per pound of turbine exhaust flow would be approximately 887 B.T.U. at an average condensing temperature of 120°F. The heat absorption capacity of the water reservoir would be approximately 130 B.T.U. per pound, assuming some heat transfer into the surrounding ground. Accordingly for each pound of exhaust steam flow through the condenser, roughly, 6.8 lbs. equal to 0.109 cu. ft. of water would be required. For the production of one net megawatt-hour of electrical energy, underground

water storage of 18,500 gallons is indicated. It is evident that this heat rejection cycle would require an extremely large reservoir capacity for any sustained button-up period.

A large reduction in the water storage requirement could be effected by permitting the turbine to exhaust its steam to the atmosphere following an attack. Operating under this condition only feed water would need to be stored. At higher exhaust back pressures, plant efficiency decreases resulting in a thermal power increase from 7.3 to 11.3 megawatts, with feed water at 60°F. The required water storage for the production of one megawatt per hour of electrical energy would be 4040 gallons. This scheme however, introduces an element of vulnerability in that it requires a breach to the surface.

Still another alternative heat rejection process, which does not require a breach to the surface, might be employed. The principal component of the process is a barometric type condenser located adjacent to the turbine. The turbine exhaust steam and water from the heat sink would mix directly in the condenser thereby providing a zero terminal temperature differential between the turbine exhaust and the stored water. With this arrangement the maximum water temperature in the heat sink can approach the boiling point. The plant efficiency would remain at the normal design level until the heat sink temperature exceeded 140°F, at which point the back pressure would gradually increase to one atmosphere. The required water storage for the production of one net megawatt hour of electrical energy would be 16,000 gallons.

The final determination of the arrangement of the plant and auxiliaries for any specific facility would be dependent on the combination of such factors as, 1) length of button-up period, 2) cost of creating water storage, 3) practicability of venting the surface.

The use of ice in the heat sink could naturally minimize the storage volume required. However, the feasibility of making, storing, and maintaining large quantities of ice in an underground installation is yet to be demonstrated. In addition, the use of ice for the heat sink would preclude the full restoration of the facility within a reasonable period of time after a button-up period.

With respect to a closed-cycle reactor powered gas turbine, the use of ebullient cooling in the precooler would allow venting the rejected heat, in the form of low pressure steam, to the atmosphere. The plant efficiency could be maintained by increasing turbine inlet conditions accordingly to compensate for the higher precooler temperatures. This operating condition could be tolerated for the duration of the button-up period. Figure 5-11 illustrates in very simplified form, the cycle diagram for this type of nuclear plant.

The ratio of power to weight and power to volume, for the pressurized water reactor plant, is quite low. This would be a more prominent disadvantage in a cut-and-cover type structure than in a deep underground

facility. The gas-cooled reactor plant has a much more favorable ratio. With regard to availability, pressurized reactors are considerably more advanced in development, and in fact, in the sizes considered applicable, could be made available for power generation within 24 months after placement of an order. On the other hand, gas-cooled reactors operating at high temperatures with enriched fuel, are not nearly so well developed and their immediate availability is questionable. Table 5.3 presents comparative characteristics of these two types of plants.

d. Other Power Sources. In addition to nuclear energy there are other means of generating electricity which are not dependent on air for combustion. The three most promising are fuel cells which employ the principle of direct conversion of chemical energy into electrical energy, and thermoelectric and thermionic generators which employ the principle of direct conversion of thermal energy into electrical energy. A characteristic of these direct conversion generators is their production of direct current at low voltage. Since the predominant utilization for hardened facilities is alternating current, some means to convert the direct current to alternating current would be required. This can be done by means of nonrotating direct to alternating current transformers which are readily available. Such converters are solid-state devices, compact and reliable, requiring little or no attention. The electrical energy produced by individual elements of these types of generators has a potential which can range from 50 MV to 3 volts. Higher voltages can be obtained by connecting individual elements in series.

Fuel cells are devices used to convert the free energy of a chemical reaction directly to electrical energy by an electro-chemical process. Although several types of fuel cells have been developed and others are under development, they all exhibit some basic similarities to the hydrogen-oxygen type illustrated in a greatly simplified form in Fig. 5-12. A brief description of the principle of operation accompanies the illustration. Fuel cells have no Carnot-cycle limitation to their energy conversion efficiency and at present operate with a one volt cell voltage and a conversion efficiency of 75 percent. Most of the rejected heat is absorbed by the electrolyte which can be dissipated to the incoming gaseous fuels. The remainder manifests itself as water vapor and radiated heat from exchangers, pumps, and cell container. Although small units up to 10 kw are presently available, there is a distinct possibility that larger units up to one megawatt may be available within five years. Attention is directed to a comprehensive report (Ref. 5.7-6) on the status of fuel cell systems which are considered promising by the Department of Defense.

Thermoelectric and thermionic generators are two types of devices capable of producing electric power directly from a heat source. Both generators are static devices and have the characteristic of the working fluid being "heated electrons," emitted into a solid in the case of thermoelectric generator and into a vacuum for the thermionic generator. Schematic drawings of each type of generator, accompanied by a brief description of the principle of operation, are shown in Figs. 5-13 and 5-14. For use in protective construction, the source of thermal energy could be either the decay heat produced by radioisotopes or by the nuclear fission process in a reactor. The potential

power supply from isotopic energy sources is extremely limited in capacity and could be considered only as a secondary supply. An example might be intrusion alarms and instrumentation located remote from the primary facility, powered by thermoelectric generators placed in hardened manholes in lieu of extending feeders from the main power source. As a primary power supply, nuclear powered generators show more promise. Conversion efficiencies at present are on the order of 5 to 20 percent, with thermionic generators at the higher range of the scale; however, efficiencies approaching 30 to 35 percent have been predicted for the future. Since both of these conversion methods require a significant heat input during their operation, and the cycle efficiencies at best are on the order of 35 percent, a heat sink of some magnitude is indicated.

The possible advantages for protective construction in employing static, low maintenance, compact power sources which do not require air for combustion, are clearly evident. Advancing technology in the past five years has produced several ways to generate electrical power by direct conversion at acceptable efficiencies, but there are still several major problems to overcome. All the systems under consideration produce a low voltage direct current power supply with the undesirable feature of the voltage dropping as the current increases. In the case of fuel cells, fuel supply, particularly for a continuously operating facility, might prove to be a costly factor. Thermoelectric and thermionic generators, because of their relatively low conversion efficiencies retain the problem of rejected heat disposal. With respect to the economic feasibility, realistic cost figures are presently unavailable.

5.7.2.4 Distribution and Regulation. There are no unusual problems connected with the distribution of power throughout a hardened facility. Established design criteria as outlined in the various Corps of Engineers and Air Force manuals (Refs. 5.7-7 and 5.7-8), should be adhered to where applicable. Special consideration must be given to minimizing to the greatest possible extent the interruption of the power supply to critical equipment. This probably will entail the duplication of feeders and substations. Every effort must be made by the Using Agency and design group to establish the magnitude and location of the critical power loads.

The extensive use of electronic data processing and electrical communication equipment in military facilities has imposed stringent requirements on the character of the power supply. Although self-generation disposes of the problem of power reliability, it creates some problems in connection with regulation of the character of the power supply. Obviously, an underground power plant adequately sized to provide firm capacity is still considered small relative to the load when compared to a public utility. The huge flywheel effect of a large utility network which can readily absorb load transients is not in evidence with an underground plant. Consequently in the design phase, emphasis must be placed on the selection of motor driven equipment and appropriate controls to insure that transient electric disturbances can be kept to a minimum. The use of static excitation and sensitive fast response voltage regulators should be given consideration. Specially designed generators having low transient reaction should also be investigated.

For frequency regulation, the use of load sensing governors offers great possibilities. With power generated from an isolated plant, a system of time error compensation would probably be required.

5.7.3 Heat Sink. In order to maintain environmental conditions suitable for the operational activities of a hardened facility, all the heat generated within must be dissipated in some manner. For this purpose an adequate heat sink must be available. As considered herein, a heat sink may be defined as a body of high thermal capacity relative to the rate at which heat is transferred to it.

5.7.3.1 General. The rock or earth surrounding an underground facility is a natural heat sink which has some capacity for heat absorption. Thermodynamic and heat transfer problems dealing with the heat absorption capability of geological formations surrounding underground structures, have been under investigation by the National Bureau of Standards since 1950. Their findings and conclusions are presented in a Corps of Engineers Design Manual (Ref. 5.7-1). It is to be noted from these findings that the rate at which the surrounding ground can accept heat is limited and decreases with time. In almost all instances the transfer of heat to the surrounding ground will not dissipate all the heat rejected from electronic equipment, building services, power generating equipment, and human metabolism. The heat generated by the electronic equipment, etc., is dissipated to the space environment, resulting in the need to integrate a refrigeration cycle into the heat absorption system. Regardless of whether an absorption or vapor compression refrigeration cycle is employed, the temperature at which the heat is transferred to the heat sink is relatively low. The problem becomes one of providing an additional heat sink into which that portion of the heat rejected from the power generation and air conditioning refrigeration equipment, which would not be absorbed by the surrounding geological formation, can be dissipated.

5.7.3.2 Design Considerations. There are several important factors which should be considered in the solution of the heat sink problem, namely, heat load characteristics, level of protection, length of button-up period, restoration capabilities, and geographic location.

a. Heat Load Characteristics. Both the rate at which heat is rejected from the power generation and air conditioning refrigeration equipment and the temperature of the heat transfer process have a profound effect on the selection of the proper type of heat sink. Cooling towers or spray ponds must be sized to receive the rejected heat at the rates produced during periods of peak operation, regardless of the length of the button-up period. On the other hand, the size of a heat absorbing reservoir is determined by the total heat rejected over the entire button-up period. With respect to the temperature of the heat transfer process, it is obvious that the maintenance of a large temperature differential between the heat sink and the power and refrigeration cycles would be highly desirable. Considering the fact that the power generation and refrigeration cycles are subject to Carnot-cycle efficiency limitations, an increase in exhaust pressures and condensing temperatures would result in lower plant thermal efficiencies and consequently a larger quantity of rejected heat. This interdependence of the

heat sink and the power plant equipment requires parametric studies in order to arrive at the best method of heat disposal. An example would be increasing the condensing temperature of a vapor compression refrigeration cycle from 100°F to 110°F with a corresponding decrease in coefficient of performance from 5.0 to 4.0. To absorb one B.T.U. the required motor energy input to the compressor would increase from 0.82 KW to 1.02 KW. Because of inherent thermodynamic cycle inefficiency, 25 to 35 percent of the increase in KW requirements would be rejected to the heat sink. This indicates that an attempt to increase the temperature differential between the heat sink and the power and refrigeration cycles by increasing condensing temperature must be carefully analyzed with respect to its influence on power generation requirements and possible increases in rejected heat.

b. Level of Protection. The consideration of the level of protection to be afforded a facility is highly important in determining the type of heat sink required. For a facility placed deep underground in order to withstand all the effects associated with very high yield weapons, there would be a compelling desire to minimize surface connections. Consideration therefore must be given to an underground heat reservoir as one solution to the problem. On the other hand confidence has been gained in the reliability of blast closures in light of recent advances in the state of the art and accordingly protected underground cooling towers using outside air as the heat sink appear to be feasible and should be given consideration.

For cut-and-cover structures the cost of creating reservoir capacity is relatively high and the use of outside air implemented by cooling towers and spray ponds should be investigated. Consistency in the level of protection for all the features of an installation dictates that such cooling towers or spray ponds must be designed to resist the same weapon effects as the facility itself. This is feasible with spray ponds in the case of moderate overpressures and there are preliminary designs in existence. As far as cooling towers are concerned, it would appear that they should be located in the protected area of the structure.

c. Length of Button-Up Period. Primarily the significance of this factor is its impact on the storage requirements of water, whether it is used as the heat sink itself, or as make-up water for cooling towers.

d. Restoration Capability. There is sometimes a requirement for restoring the mission capability of a facility in as short a time as possible after the termination of the button-up period. If such restoration necessitates the refilling of an underground reservoir, serious consideration must be given to some other type of hardened heat sink.

e. Geographic Location. It is obvious that the geographic location of a facility will determine the climate and conditions of outside air and perhaps the prevalence of underground water for heat removal. This leads to the consideration that geographic locations which offer the possibility of developing an underground water supply within the facility itself should be given consideration.

5.7.3.3 Type of Heat Sinks. Basically there are only two heat absorbing substances which deserve serious consideration at the present time. These are outside air and water either in its liquid form or as ice or a combination of both. Heat absorption in any large quantities by other chemical substances is in the research phase and considerable development work is still to be done to determine feasibility. In general an adequate water supply would be available at or near a facility location for use prior to an attack. If the water source for such a supply is a natural stream of continuous flow adequate for cooling purposes there is no necessity for considering the conservation of such a supply. On the other hand, if the supply is from an underground aquifer, consideration should be given to conserving such a supply during normal operation by the use of cooling towers or spray ponds located at a convenient place outside the facility. It must be recognized that such cooling towers or spray ponds and the pipeline connection from the water source to the facility would probably be destroyed during an attack and therefore some form of hardened heat sink not dependent on an outside water supply must be provided.

a. Protected Water Supply. It is apparent that the most desirable means of heat dissipation would be a supply of water developed within the protected area adequate for use in a once-through cooling cycle during all operational conditions. Such a water supply might be either an underground aquifer below the protected area or an infiltration of water through the surrounding geological formation into that area. Although the flow rate of such a supply might not meet peak load requirements, a retention reservoir could be utilized to meet the peaks. Consideration should be given to conserving the underground supply for use following an attack. However, it must be recognized that the possibility of finding an adequate water supply from a source in the geological formation immediately surrounding the installation, will occur only in rare instances and, in general, other means of heat dissipation must be investigated even if only to supplement an inadequate source that may be available within the protected area.

b. Underground Reservoirs. An underground reservoir for the storage of a heat absorbing substance has the distinct advantage of not requiring any breaches to the surface. A number of theoretical and experimental investigations have been made to determine the performance of various substances and their containment when utilized as heat sinks. Such substances must have heat absorbing characteristics that are compatible with the heat rejection processes of the power plant and refrigeration equipment. Three types of heat absorbing substances will be considered, namely water, ice, and chemicals.

1. Water. Compared to ice and chemical substances, water has a relatively low heat absorbing capacity per unit volume. Assuming an initial water temperature of 50°F and a rise in temperature to 100°F, a cubic foot of water would be capable of absorbing 3,120 B.T.U.

For a refrigeration cycle the maximum water temperature should be held to 100°F as governed by the heat transfer process of a vapor compression

cycle. Higher temperatures would significantly reduce the coefficient of performance and might increase maintenance and operating problems due to the correspondingly high condensing pressures. An absorption cycle imposes even more stringent requirements on the allowable temperature rise. Power generating equipment on the other hand can tolerate water temperatures up to 200°F.

As previously discussed, the geological formation surrounding an underground reservoir is capable of absorbing a considerable amount of heat, particularly if the temperature differential between the stored water and the surrounding earth or rock is substantial. This phenomena should be taken into consideration in the design of the heat sink. It is therefore highly desirable to permit the stored water to reach as high a temperature as possible. Since the allowable temperature necessary for refrigeration and power generation equipment differ so significantly, it is mandatory that consideration be given to providing separate reservoirs. Such an arrangement does not entail any significant increase in cost.

In addition, serious consideration should be given to precooling the stored water to a temperature of about 40°F. This would also lower the temperature of the surrounding geological formation below its initial level. This pre-cooling could be accomplished prior to occupancy by the refrigeration equipment installed for air-conditioning purposes and could be maintained after occupancy by the same equipment during off-peak operating periods. In the beginning of a button-up period the pre-cooled water could be circulated directly through air-conditioning cooling coils, thus postponing the need to operate the refrigeration cycle. Such an arrangement should result in substantial savings in electric power consumption and a corresponding reduction in the total amount of rejected heat. Equations to calculate the required capacity of an underground reservoir to be used as a heat sink are presented in a Corps of Engineers Design Manual (Ref. 5.7-1), and sample problems are also presented in this manual.

After an attack it is conceivable that the pipeline connections from the basic water source to the reservoir could be repaired in the event they were damaged by an attack. As soon as such repairs are effected the water reservoir could easily be refilled for further use of the facility.

Studies made by the National bureau of Standards (Ref. 5.7-9) indicate that the heat transfer from water to rock is proportional to the relationship of contact surface to volume. Theoretically, the heat absorption capability of the rock increases as the relationship of contact surface to volume increases. This leads toward the consideration of reservoir excavation with a relatively small cross section. On the other hand, economical excavation methods must be taken into consideration in determining the cross section.

2. Ice. The fact that melting ice has a relatively high heat absorption capability makes the consideration of its use extremely attractive. For example, a mixture of 50 percent ice and water with an original temperature of 32°F and a rise to 100°F would absorb 8400 B.T.U per cubic

foot as compared to 3120 B.T.U. for water alone. This would indicate that, at least theoretically, the volume of a 50 percent ice and water reservoir could be considerably less than one half that required for a reservoir containing water only.

Although some experiments on a small scale have been conducted by the Bureau of Standards, no feasible reliable method has yet been devised for the distribution of manufactured ice in a large reservoir. Another problem which has not been solved is the maintenance of the ice content in the reservoir after it has been filled. The mechanical equipment required to store and maintain the ice could involve large expenditures. The technical and economic feasibility of utilizing ice in the reservoir of a specific underground facility has been presented in a comprehensive report (Ref. 5.7-10). The findings of this report indicated that it was more economical and far more reliable to excavate the additional water storage capacity rather than to utilize ice in such a reservoir. In addition, the use of ice in the heat sink would preclude the possibility of restoration capability in any reasonable time following a button-up period since it would probably take several months to re-establish the required ice content.

3. Chemical Substances. Research is being conducted by several organizations dealing with the feasibility of using various types of reversible endothermic processes for utilization as heat reservoirs. Primarily the application of this process is being directed toward the field of space heating and nuclear-steam power. It is conceivable, however, that it could be utilized as a hardened heat sink in the field of protective construction. Available data gives evidence that of the known reversible endothermic processes, the "heat of fusion" of salt-hydrates and salt-ammoniates appear to offer the highest heat absorbing capacities per unit volume. These salts are soluble in either water or liquid ammonia and consequently the sink could be in a liquid state. Assuming a heat sink temperature rise from 50°F to 100°F, heat absorption capacities on the order of 20,000 B.T.U. per cubic foot are indicated. However, considerably development work is still required to bring into practical realization a chemical heat sink applicable to the type of facilities under consideration in this manual.

c. Outside Air with Cooling Towers. The atmosphere itself provides a heat sink of unlimited capacity, and therefore its use should be considered in the design of a heat sink for a hardened facility.

The heat rejection equipment required to implement the air is either some type of cooling tower or a spray pond. With respect to cooling towers, consideration should be given to the open filled type, the closed prime surface evaporative type, and the closed extended surface dry type. All of these types are extremely vulnerable to blast pressure and if required to survive an attack and then function during the button-up period, they must be placed underground in the protected area.

1. Open Filled Type Cooling Towers. This type of equipment is the most efficient of the heat transfer devices mentioned above due to the closer terminal temperature difference between the outside air and the

cooling water. It does require a protected water storage capacity for make-up purposes during a button-up period. However, the use of this type of cooling tower introduces a potential contamination hazard unless the air required is passed through filtering devices designed to remove radioactive particles. Without such filtering these particles would precipitate out into the cooling tower basin and in turn would be circulated with the cooling water through the power generating and air-conditioning refrigeration equipment resulting in the possible build-up of a dangerous concentration of radioactive matter. For relatively large installations which would require considerable quantities of outside air, the cost of providing the necessary filters and the additional power to move the air through such filters might very well indicate that the use of this type of cooling tower is uneconomical.

2. Closed Prime Surface Evaporative Type Cooling Towers. The use of this type of heat rejection equipment would eliminate the contamination hazard in that no radioactive particles would enter the circulating system. The heat transfer efficiency is only slightly lower than for the open filled type resulting in an increase of about 10 percent in the quantity of outside air required. It is evident that this increase is insignificant in the light of the advantage to be gained in the elimination of the radiation hazard and its attendant requirement for filtering devices. This type of cooling tower also requires water storage for make-up purposes.

3. Closed Extended Dry Type Cooling Towers. The primary disadvantage of this type of heat transfer equipment is the relatively enormous amounts of outside air required for its proper functioning. This may be on the order of ten times more than either of the types mentioned previously. This type of equipment also eliminates the contamination hazard and it offers the added advantage of not requiring any storage of water for make-up purposes. The possible saving thus effected, however, must be balanced against the increased cost of providing relatively large size surface connections for air intake and exhaust including the blast closures for these openings, not to mention the potential increase in vulnerability to blast damage due to these large openings. It should be noted however, that for a small installation using cut-and-cover type structures and with refrigeration loads on the order of 100 tons and electric power demands of 500 KW or less, this type of equipment should receive serious consideration. In such instances the cost of creating water storage for make-up purposes might very well be greater than the expenditures involved in providing larger surface connections for outside air circulation.

Recent comparative studies made for two large deep underground installations indicate that the utilization of closed prime surface evaporative type cooling towers located in the underground behind blast barriers proved to be the best solution for the heat sink problem.

In connection with the utilization of underground cooling towers consideration should be given to their possible use as a method of absorbing heat from the power plant and building spaces. Two arrangements merit a detailed study and investigation to determine the best selection for a specific

facility and site location. One would make use of the outside air to ventilate and absorb heat from the power plant space before passing through the cooling towers. Parametric studies should be made to determine the net effect on the increase in the total heat of the air as it cools the power plant. The costs attendant to filtering the outside air before introducing it into the power plant must be taken into consideration and may negate other advantages to be gained by employing this arrangement. The other arrangement would utilize the cooling water circulating through the tower as a heat absorbing medium. During warm weather, the cooling water can be used only for power plant cooling, inasmuch as its temperature would be higher than the building space temperature. During colder weather, however, the cooling water would be at a sufficiently low temperature to absorb heat from both the power plant and building spaces. This arrangement offers two significant advantages. First, it obviates the need for an extensive air filtration installation and secondly, it is capable of absorbing heat from the building spaces resulting in a reduction in the running time of the mechanical refrigeration equipment and consequently a decrease in the total electrical energy consumption. Although the temperature of the cooling water leaving the tower would have to be somewhat lower, in order to cool the power plant, the net adverse effect on the heat sink is not nearly as significant as in the case of power plant cooling with outside air.

d. Outside Air with a Spray Pond. A spray pond can be designed with some degree of hardness and is therefore applicable in connection with facilities which require resistance only to relatively low blast pressures. On the other hand, a spray pond is far less efficient than any type of cooling tower and is subject to the vagaries of the prevailing winds due to its exposed location. This exposed location also makes it highly susceptible to radioactive contamination. The use of a spray pond for heat transfer in connection with facilities of high level protection is certainly questionable in view of the surface distortions and ground shock associated with high yield weapon detonation which would probably render the spray pond inoperable.

5.7.4 Air Supply and Conditioning. Air supply requirements for a hardened facility derive from the necessity to introduce fresh air to replace that vitiated by personnel and vehicular traffic, combustion air for power generating equipment and other air aspirating devices and an air flow for heat absorption. That portion of the air supply used for ventilating purposes must be conditioned to maintain the proper environmental conditions for personnel activity and special electronic equipment ventilation.

5.7.4 1 General. It is clear that the introduction of outside air into a hardened facility necessitates breaching the protective structure in some manner. Such openings must be provided with quick closing blast resistant devices which are susceptible to malfunction and therefore introduce points of weakness in what may otherwise be a relatively invulnerable facility. In view of this, it is highly important to investigate thoroughly the air supply problem with a view towards minimizing requirements particularly during the Post-Attack operational period.

The best method of introduction, subsequent treatment, distribution and exhaust of outside air passing through a hardened facility may be different

for the three operational phases, namely, prior to, during, and following an attack. In addition the specified level of protection, the configuration, and means of access must be taken into consideration in the design of the various features of the air supply systems. The possible variations in all these factors make it difficult to set forth a typical design. It may be stated in general, however, that the introduction of an air supply into a protected facility and its subsequent exhaust into the outside atmosphere may involve large expenditures for connections to the surface and the protection of those openings against the blast pressures associated with a nuclear weapon attack. For this reason serious consideration must be given to every possible multi-purpose use of the air supply.

5.7.4.2 <u>Ventilation</u>. A fresh air supply for ventilation purposes in a hardened facility is required to maintain habitable conditions for personnel and purge access tunnels in the event they are used for vehicular traffic.

a. <u>Personnel</u>. It is normal practice to supply fresh air to commercial or public buildings in an amount sufficient to dissipate unpleasant odors from persons and other processes due to occupancy, and also adequate to insure conditions conducive to normal physical activity. For a hardened facility any outside air which is introduced to areas of human occupancy must be treated in some manner to eliminate the possibility of carrying C.B.R. contamination to those areas. Criteria for fresh air quantities are generally based on per capita requirements and it is obvious for the proper design of the ventilation air supply system, the personnel occupancy during the prior- and post-attack periods must be established as closely as possible. Careful consideration should be given by the using agency to the possibility that the population during the alert and post-attack periods might be considerably in excess of that required for the prior attack operation of the facility. Personnel requirements for 24-hour operation must be determined and it is quite possible that security personnel, such as outside guards, might have to be housed during button-up periods.

It is normal practice to supply fresh air in an amount of 10 to 20 cfm per person based on the expected population. There appears to be no reason to vary this amount for hardened facilities unless the exhaust air requirements from such areas as toilet rooms, kitchens, decontamination, special processing, etc., are unusually heavy. However, during the button-up period power consumption and related heat rejection may be minimized by reducing fresh air supply to 3 cfm per person. This amount would limit the carbon dioxide content to an acceptable 1 percent for an indefinite period. It would also be more than adequate to maintain the oxygen content at the normal level of 21 percent.

Air quantities for the proper ventilation of kitchens and toilet rooms are established in appropriate Corps of Engineers and Air Force Design Manuals. (Refs. 5.7-1 and 5.7-7) These criteria are applicable for protective construction. In order to reduce the fresh air supply to an amount governed by population, the exhaust from toilet areas can be passed through activated carbon filters to reduce odor concentration and then recirculated

through the system. On the other hand, however, exhaust air from kitchen areas should not be recirculated.

The ventilation air supply entering the facility following an attack must be filtered to remove radioactive particles prior to its introduction to occupied spaces. Furthermore, it is possible to contaminate the air supply at any time by covert sabotage with chemical or biological agents which are extremely difficult to detect. The only positive means of protection is to filter the incoming air continuously. The U. S. Army Chemical Corps has designed and manufactures filters which will arrest C.B.R. contaminants from the air supply and they are available in various sizes for installation in hardened facilities. In order to conserve the capability of these special filters during nonemergency periods, standard commercial dust filters should be provided on the upstream side. Filters are inherently unable to withstand any appreciable pressures and therefore must be located on the protected side of the blast barriers.

Immediately following an attack, the outside air would have an extremely high content of radioactive particles and in order to prevent a heavy concentration of such particles in the filters, the ventilation air supply should be cut off entirely for a period of 4 to 8 hours. Since the volume of air per person within the protected area is generally relatively large, carbon dioxide pollution and oxygen contents will not reach dangerous limits during such periods. It is of course possible to revitalize the air chemically for a prolonged button-up period after an attack in lieu of re-establishing the fresh air supply.

During the button-up period there exists the possibility of an infiltration of C.B.R. contaminants around the equipment, personnel, and vehicular access doors. In addition there may be some small unknown openings which may be subject to infiltration also. Infiltration around the doors can be minimized by incorporating gas seals in the design of the doors. However, such seals are sometimes difficult to maintain tight. Exhaust valves for the passage of vitiated air from the protected space to the outside must of course be kept in open position except for the short period during and immediately following the blast. In order to prevent the possible infiltration of airborne C.B.R. contaminants after the exhaust valves are reopened, an internal pressure above the ambient atmospheric should be maintained. This can be accomplished by the use of pressure differential sensing devices which would regulate air volume devices in the exhaust openings. An internal pressure of 0.5" of water above atmospheric is sufficient to prevent infiltration and provide sufficient pressure for air lock scavenging of decontamination areas. With proper regulation of exhaust air volume the quantity of outside air required for personnel should be adequate to establish and maintain the necessary internal pressurization. A comprehensive discussion of the problems involved in the protection of structures against C.B.R. contamination is presented in Ref. 5.7-11.

b. Vehicular Traffic. In general the access to shallow cut and cover hardened facilities will be by means of relatively short tunnels or vertical shafts with nominal vehicular traffic if any. On the other hand,

the access to deep underground installations in a mountain side will be by means of a long tunnel frequently with two portals widely separated. The servicing of such a facility with personnel and supplies, particularly during pre-emergency periods, may require the use of gasoline and diesel fuel burning vehicles. If possible, the access tunnel should also be made to serve as a fresh air intake and vitiated air exhaust by the installation of a frangible barrier across the tunnel at an appropriate location to separate the intake and exhaust air flow. In general, the vehicular access portion of the tunnel would be used for exhaust purposes. Calculations have indicated that unless the expected vehicular traffic is unusually heavy the quantity of vitiated air normally exhausted from the protected space into the access tunnel will be adequate to limit the concentration of CO to an acceptable 0.04 percent by volume. CO detection equipment should be installed to monitor the air in the vehicular portion of the tunnel. In order to assure an additional air supply in the event actual operations result in undesirable haziness or excessive CO content, booster fans should be provided in the frangible barrier to exhaust from the fresh air intake side to the exhaust side.

5.7.4.3 <u>Combustion Air</u>. An uninterruptable power supply is generally required for a hardened facility and if power is being generated by diesel engine or gas turbine equipment, combustion air must be available at all times and positive means for continuous exhaustion of the combustion gases also must be provided.

Immediately following an attack the outside air is expected to contain a considerable amount of radioactive matter, a large percentage of carbon monoxide and be at a relatively high temperature. While such atmospheric conditions would make the outside air unsuitable for personnel consumption, general ventilation and heat dissipation purposes, it still would be satisfactory for combustion uses. This indicates that a closed conduit system for combustion air and exhaust is highly desirable since it would permit continuous air flow to and from the prime movers except during the blast phase itself without danger of contaminating the occupied spaces. In order to protect such a conduit system from nuclear detonation effects it must be provided with blast resistant closures and auxiliary valves at intake and exhaust arranged to insure continuing flow during the blast phase. Further discussion of such devices is presented in Para. 5.7.4.7.

5.7.4.4 <u>Heat Dissipation</u>. It is evident that a refrigeration cycle must be relied upon for space cooling purposes during periods when the outside temperature is higher than 60°F. However, when that temperature is lower than 60°F it is theoretically possible to accomplish space cooling by circulating outside air directly through the space, thus eliminating the necessity for operating the refrigeration cycle, with resultant reductions in power consumption and other operating costs. The desirability of installing a system capable of introducing sufficient quantities of outside air for space cooling purposes must be based on an economic study taking into account operating costs and capital expenditures for intake and exhaust shafts, blast closures and air filtration equipment. For large, deep underground installations the high capital cost will probably offset the savings in operating costs that could be realized by effecting air conditioning without

refrigeration during favorable atmospheric conditions. On the other hand, however, for small, shallow cut-and-cover or partially buried structures, the application of this principle has more merit and certainly should be investigated.

It is recognized that the power plant area and possibly some other equipment spaces can tolerate ambient temperatures in the range of 100°F to 110°F without seriously affecting the operation of the equipment. This suggests the possibility of utilizing outside air as a cooling medium without a refrigeration cycle at practically all times unless the geographic location of the facility is such that very high outside ambient temperatures prevail. At any location, however, the outside air would be unsuitable for this purpose for some period following an attack and therefore a supplemental cooling system would have to be considered for use during this period. Another alternative in lieu of mechanical refrigeration for space cooling would be the use of fan-coil recirculating units in conjunction with water from the cooling towers or underground storage passing through such units prior to its introduction to the air-conditioning refrigeration and power generation equipment heat rejection system. The determination of the most desirable system requires investigation of technical feasibility and comparative cost studies.

5.7.4.5 Multi-Purpose Use. Due to the interdependence of the utility components the feasibility and economics of multi-purpose uses of outside air must be carefully analyzed to assure the proper choice and design of the air supply system. In making such studies several parameters must be considered as follows:

(a) The dry and web bulb temperature of the air entering the underground cooling towers.
(b) The condensing temperature of the refrigeration cycle.
(c) The power plant area space temperature.
(d) The heat transfer surface of the cooling towers.

The items governed by and plotted against these parameters should include the following:

1. Quantity of air required for the heat sink expressed in cfm.
2. Coefficient of performance of the refrigeration cycle expressed in KW/ton.
3. Power required to move the required cfm expressed in KW.
4. Costs of air shafts and blast closures.
5. Costs of power generation, refrigeration, and air filtration equipment.

5.7.4.6 Air Conditioning. In general the specific temperature, humidity and other air conditions required in underground facilities may not be different from those maintained in "soft" surface structures when the uses of the spaces are similar. Considering the fact that the hardened facility will probably be constructed underground or partially buried, the

installation will not be exposed to the variations in climatic conditions as is a structure on the surface, except for the temperature variations of the outside air supply. Although seepage water could affect the space humidity conditions, the amount of water to be encountered underground is extremely difficult to predict. Past experience, however, gives evidence that little or no adverse effect should be expected even though initially the excavation may be considered "wet". This is particularly true of underground excavations housing free standing internal structures. Not to be overlooked, however, is the net effect on the air conditioning system of the possible absorption of seepage water by the space air. Since the process is essentially an adiabatic saturation, the total heat remains unchanged, that is, the gain in moisture is obtained at the expense of a corresponding drop in dry bulb temperature.

From the foregoing discussion it becomes apparent that the heat loads which significantly affect the air conditioning system, particularly the refrigeration cycle, are those contributed by the outside air supply and the rejected heat from people, lights and electrical equipment. Inasmuch as the utility problems are greatly magnified by the imposition of the refrigeration cycle, every effort must be exercised to reduce the outside air supply for ventilation and the net electrical power needs to a minimum. Transistorized electronic equipment with cooling should be utilized wherever possible and lighting should be of the gas discharge type such as fluorescents.

In regard to the spaces housing the "hardware", the selection of interior air conditions are invariably governed by criteria established by the system contractor. The conditions are suitable for personnel efficiency and are usually within the practicable range attainable with conventional air-conditioning equipment.

Since comfort is not always a prime objective, the designer is faced with considerable latitude in the range of temperatures and humidities that can be selected for the interior general work areas. Experience has shown that personnel can sustain a considerable range of temperatures without serious loss of efficiency.

Some of the factors that influence the selection, for design purposes, of an interior air condition are listed below:

a. In regard to transmission and solar heat gains, the direct effect of climate on the interior environmental conditions may be considered inconsequential since the installation under consideration generally will be placed partially or entirely below ground.

b. Since the rock or earth mass surrounding the installation is at or near the mean annual temperature of the site, partially occupied interior spaces may require the addition of heat to maintain a reasonable degree of comfort.

c. Regardless of the outdoor conditions, the heat generated by personnel, lighting and other internal loads will dictate the need for cooling during full operation.

d. The selection of a higher space temperature makes possible the utilization of a greater differential in temperature between the space and conditioned air supply. Consequently, a reduction in the physical size of the air handling equipment and distribution ducts may be possible.

In view of the foregoing, it appears reasonable to select an interior air condition of 75°F ± 3°F and relative humidity not exceeding 55 percent as an acceptable compromise between the often recommended 70°F and 80°F indoor temperatures during the winter and summer seasons respectively.

During stand-by operation, the unoccupied spaces are cooled by the surrounding rock and earth mass and under this circumstance heat may be required to maintain a suitable relative humidity for material preservation. Assuming the natural air conditions in the underground installation at 55°F and nearly saturated, an increase in the interior space temperature to 70°F will reduce the relative humidity to about 55 percent which is considered satisfactory for the preservation of most materials and equipment.

In regard to the refrigeration cycle, consideration should be given to the vapor compression and absorption principles. For small compact facilities, extended surface heat exchangers with direct expansion of the refrigerant may be used to dissipate the space heat. Large facilities may find the circulation of chilled water through the heat exchangers to be a more feasible solution. In the case of the vapor compression cycle, the refrigerant gas temperature is elevated to a practical level for condensing purposes in reciprocating or centrifugal compressors driven by electric motors or steam turbines. The principle of the absorption cycle is the self cooling of the chilled water by flashing in a high vacuum in the presence of a lithium bromide solution. Steam or hot water is used to maintain the concentration of the lithium bromide solution. Commercially available standard sizes can be selected to fulfill the cooling requirements of any of the facilities under consideration.

From the previous discussions on power generation it becomes apparent that the final selection of the refrigeration cycle might be largely dependent upon obtaining a combination of power generation and refrigeration equipment which will produce the lowest total heat rejection with the view towards reducing to a minimum the heat sink requirements.

An example might be the case of an open cycle gas turbine power plant employing the economizer to recover heat from the exhaust gases in the form of low pressure steam or hot water for use with an absorption refrigeration unit. This utility arrangement will increase the plant thermal efficiency to the extent of producing a ton of refrigeration without increasing the power generating capacity. The heat rejected from the refrigeration cycle will be on the order of 30,000 BTU/HR/Ton. On the other hand, if a vapor compression unit, electric motor driven, is used, the heat rejected will be on the order of 15,000 BTU/HR/Ton and the plant generating capacity increased by about 1 KW/Ton. Depending on the factors of geographic location, degree of hardness and operational condition, the increased cost of the heat sink for the absorption cycle may override the cost of the enlarged power plant.

Obviously the number of possible combinations of power generation, refrigeration and heat rejection equipment is virtually unlimited. Some may be eliminated from further consideration merely by inspection, other possibilities may have to be investigated thoroughly. In the final determination of the type of refrigeration cycle to be adopted, emphasis must be placed on the interdependence of the principal utility components and their impact on the over-all power and heat rejection loads. Only then can the best solution to the problem be attained.

Past experience with facilities of the types under consideration indicate an ever changing utilization of the available space within the protected areas. Usually the changes occasion a major modification of the air conditioning system. In recognition of this, it is incumbent upon the Design Engineer to provide an air conditioning system that will accommodate a nominal redistribution of internal loads but not necessarily with an increase in the over-all facility load. The supply of conditioned air to the building areas on a modular basis is considered highly desirable.

In regard to equipment selection, every effort must be made to utilize components that require a minimum of floor area and volume. A general advantage lies with small ducts and medium or high velocity air distribution. A limiting factor may be excessive fan horsepower or noise associated with high fan speeds and air velocities. Only incombustible materials including vapor barriers should be used.

In regard to the areas housing essential electronic and electrical equipment, the use of a duplex air conditioning system may be required to conform to the over-all reliability of the weapon system. Since the "hardware" may be a predominant feature of the hardened facility, it is essential that the service requirements imposed by the system contractor be coordinated, at the outset, with the design of the air conditioning system.

5.7.4.7 Blast Closures. All openings from the protected facility to the outside must be provided with blast closures. Those used for personnel, equipment and vehicular access can and should be closed at the start of an alert and their operation therefore can be relatively slow. The openings for air supply and exhaust on the other hand must be kept in an open position during the alert period and designed to close instantly by the blast pressure or when actuated by some sort of equipment which would be triggered by devices sensitive to nuclear effects. Such devices have been developed by the U. S. Army Signal Corps. The fresh air supply for personnel, depending on the ratio of internal volume per capita, can be interrupted for several hours without any deleterious effects. In the event underground cooling towers are employed for equipment heat disposal, provisions can be made to store water as the heat sink for a period of several hours so that the air supply to the towers may also be interrupted. In view of this the blast closures in the air shafts serving personnel and cooling towers can be arranged to remain closed following an attack and then opened manually after it has been determined that the condition of the outside atmosphere is free of heavy radiological contaminants and satisfactory in regard to temperature and carbon monoxide content.

No interruption, however, of air flow through diesel engines or gas turbines is tenable. Such equipment must aspirate and discharge combustion products continuously in order to maintain operation. Therefore, the blast closures, for combustion purposes must be reopened immediately after the subsidence of the blast pressure. However, the positive phase of a high yield weapon detonation may last as long as 3 to 5 seconds and such an interruption of air flow would result in a shutdown of the power generation equipment. For the hardened facility which must continue to function during and immediately following an attack such an interruption in power supply is not tolerable. To obviate this condition, auxiliary valves opening to the protected space must be provided in the air induction and exhaust systems. They must be designed to open instantly whenever the blast closures in the openings to the outside have been shut. The prime movers would then aspirate from the protected space, and combustion products would be diverted into the power plant area. The length of time that such a diversion can be sustained would be governed by space temperature rise and by the allowable concentrations of toxic and noxious fumes to which personnel could be exposed. In general, a diversion time of 10 seconds is tolerable which is long enough for the initial blast pressure to subside and for the subsequent reopening of the blast closures. It is self-evident that the blast closures and auxiliary valve must be arranged to actuate simultaneously.

The use of catalytic afterburners installed in the exhaust gas opening to the protected space may be considered as a means of reducing the degree of contamination during the diversion period. These devices, however, operate effectively only at elevated temperatures which would not be in evidence at the outset of the diversion cycle. Several minutes would elapse before the afterburner could function properly and in the interim the concentration of toxic and noxious fumes in the exhaust gases would remain unchanged. In view of the foregoing, no further consideration should be given to the use of these devices.

5.7.5 <u>Water Supply</u>. The requirement for water supply is generally influenced by two determining factors namely, the domestic consumption and the heat sink requirements, with the latter varying over a wide range as discussed previously in this manual. In the relatively few instances where dry-type cooling towers can be utilized for equipment heat disposal the demand for water supply is governed only by requirements for domestic consumption including fire protection.

5.7.5.1 <u>Basic Requirements</u>.

a. <u>Heat Sink</u>. The conventional rule of thumb method for estimating heat sink requirements, namely 1 to 2 gpm per ton of refrigeration, 2 to 3 gpm per KW for steam power generation and 0.2 gpm per KW for diesel power generation can be applied only if a once-through cooling cycle is utilized. Since it is unlikely that a water supply of adequate capacity for such use can be developed within the protected facility, the quantity of water required for equipment heat disposal will be governed by the type of heat sink adopted and the length of button-up period. Because of the interdependence of the utility system components and the many varying types of hardened

facilities under consideration it is impossible to present a rational quantitative analysis of the cooling water requirements, which may range from 25 to 1000 gpm.

b. Domestic Consumption. It is normal practice to supply water to a military base in the amount of 75 to 100 gpd per capita. A hardened facility, if located remote from a major support installation, would probably include billeting, messing and recreational facilities in addition to the operational elements. Accordingly a water supply for domestic purposes, prior to an attack should be based on amounts in the order of 75 to 100 gpd per capita. Following an attack the consumption of water can be limited to an amount necessary for drinking and minimal hygienic purposes in the interest of minimizing the storage requirements for the button-up period. A water use in the range of 20 to 30 gpd per capita is considered adequate.

Consideration must also be given to the process water requirements, if any, and to the fire protection needs. In the case of the former, the quantities required for certain types of facilities may have a greater influence on the selection of the water supply and storage system than would the domestic needs.

5.7.5.2 Water Sources. The adoption of the concept of a "button-up and restoration capability as soon as possible" dictates the development of some type of protected water supply. It is evident that the most economical and least vulnerable would be a source developed within the protected area of the facility. However, the probability of developing such a supply in quantities adequate for operational purposes will occur in only few instances. In general some outside source will have to be developed.

a. Well Water. There are a number of advantages in a dispersed well field located several miles from the hardened facility itself. Well houses, constructed as shallow cut-and-cover structures designed for nominal blast resistance, could be considered as hard, by virtue of dispersion, as the facility itself. In addition, a well water supply would not be susceptible to radioactive contamination. Furthermore the temperature of the water is likely to remain at or near the mean annual temperature of the surrounding area and consequently is readily usable for domestic as well as equipment heat disposal purposes.

A dispersed well supply requires buried pipe line connections to the facility, and it must be recognized that even though they have some inherent overpressure blast resistance they are a weak link particularly in the proximity of the facility itself. The facility must be self sustaining for a long enough button-up period to permit repair of any ruptured lines or the replacement of those lines with light weight P.O.L. piping installed aboveground. Conservative estimates of the time required to restore the capability of the water supply system vary from a week to several weeks.

b. Surface Water. There may be instances where the development of a well water supply is either impossible or too costly and some natural water shed would have to be utilized. The flow of the river or

mountain stream in such a water shed should be thoroughly investigated hydrologically to determine its discharge during dry seasons relative to the quantities of water required by the facility for operational purposes. It may be that even the dry season discharge would be of sufficient quantity to be used in a once-through cooling cycle. The creation of an impounded supply by means of a dam in the water shed should be avoided if at all possible, since the dam itself would be highly vulnerable to a nuclear attack and the impounded lake would be highly susceptible to contamination sabotage. If a surface supply is unavoidable, consideration must be given to the use of special water treatment to counteract the effects of radiation. It is conceivable that the entire water shed may be contaminated with long half-life radioactive substances. Although these may constitute only a minute percentage of the total fallout, continued ingestion by humans could prove fatal, and therefore, a conventional ion-exchange demineralization system should be provided in the facility. The demineralized water which is considered "aggessive" must then be further treated to make it compatible with standard plumbing and piping materials.

5.7.6 Sanitary Facilities. In general distribution of domestic water and the collection of sanitary wastes within the protected facility are of conventional design. Acceptable design procedures are set forth in appropriate military manuals (Refs. 5.7-7, 5.7-12, and 5.7-13) and should be adhered to wherever applicable. Consideration should be given to the use of chemical toilets for very small hardened satellite facilities, in order to conserve the water supply during the button-up period. However, in the larger facilities, the use of chemical toilets would be impractical and conventional means must be employed. The soil and waste stacks can be fitted with activated charcoal canisters, at their terminals, thus allowing them to vent into the protected space. The combination of charcoal and dilution by the ventilation system will render the vapors innocuous.

The collected sewage ultimately must be conveyed to the outside and disposed of. Wherever possible the protected facility should be locally sited to permit gravity drainage of the sewage. In all probability, however, cut-and-cover type structures will require the use of a lift station and force-main to conduct the sewage to the outside. The ultimate means of disposal may include existing municipal or military sewerage systems, new local treatment facilities or raw stabilization ponds..

In regard to the disposal of rubbish, kitchen run garbage and classified material, consideration must be given to some form of destruction and volume reduction. Two methods are considered feasible, namely incineration or comminution and extraction. In the case of the former combustion air is required and in the latter, greater storage is needed.

5.7.7 Fire Protection. Although the structures under consideration as well as the housed furnishings and equipment are predominately incombustible, it is extremely important to preclude the growth of incipient small fires supported by paper and similar material in areas within the structure which may be unoccupied for extended periods of time.

The use of automatic water sprinklers is considered undesirable because of the possible damage to electric and electronic equipment. It is recommended that consideration be given to the use of a fire detection system in conjunction with portable dry chemical or carbon dioxide extinguishers for fire protection. The use of occupational (1-1/2 inch) hose racks in service areas are also considered applicable. The fire detection system should be of the coded, closed circuit, supervised proprietory type.

A system of smoke abatement must be considered since the facility is without fenestration or other direct openings to the outdoors. The air conditioning system should be arranged to minimize the possibility of distributing or recirculating fire or smoke throughout the facility. A subdivision of the installation into fire zones wherein the supply of conditioned air and the exhaust of the vitiated air is balanced will tend to reduce the spread of smoke. A separate exhaust system capable of withdrawing air from any one fire zone or subdivided area and discharging the air directly to the outdoors is considered desirable. Controls can be provided interlocked with the fire detection system to energize the exhaust fan and position a damper which will connect only the affected zone or area into the exhaust system for smoke purging.

5.7.8 References.

5.7-1 "Heating and Air Conditioning of Underground Installations," Corps of Engineers, EM 1110-345-450, November 1959 (Unclassified)

5.7-2 Cross, W. H., Commander, U. S. Navy, "Utility Problems in Submarines and Their Relation to Underground Construction." Proceedings of 2nd Symposium on Protective Construction, The Rand Corp. (Unclassified)

5.7-3 Williams, D. G., Colonel U. S. Army, "The Army Nuclear Power Program," Paper No. 15, Nuclear Engineering and Science Conference, April 1960 (Unclassified)

5.7-4 Rosen, M. A. and Bicher, G. A., Captain, U. S. Army, "Army Gas-Cooled Reactor Systems Program," Paper No. 44, Nuclear Engineering and Science Conference, April 1960 (Unclassified)

5.7-5 PM-1 Nuclear Power Plant Program, The Martin Company, Contract AT(30-1)-2345, with the USAEC (Unclassified)

5.7-6 Stein, B. R., "Status Report on Fuel Cells," ARO Report No. 1, June 1959 (Unclassified)

5.7-7 "Standard Outline Specifications for Air Force Facilities," Manual No. 88-15A, Department of the Air Force, December 1959 (Unclassified)

5.7-8 "Electrical Design," Corps of Engineers, Manual for Military Construction, Part VI, Chapters 1 and 2, May 1953, (Unclassified)

5.7-9 "Some Observations on the Use of Underground Reservoirs as Heat Sinks," National Bureau of Standard, Report 4795, July 1956 (Unclassified)

5.7-10 "Icing of Underground Reservoirs," Feasibility Study, Project DOD-55-VAR, Parsons, Brinkerhoff, Quade and Douglas for the Corps of Engineers, Washington District Office, February 1958 (SECRET)

5.7-11 "Protection of Structures from CBR Contamination," U. S. Army Chemical Corps Engineering Command, ENCR No. 30, June 1959 (Unclassified)

5.7-12 "Mechanical Design," Corps of Engineers, Manual for Military Construction, Part V, Chapter 4, May 1959 (Unclassified)

5.7-13 "Sewage and Water Disposal," Corps of Engineers, Manual for Military Construction, Part VIII, June 1959 (Unclassified)

5.8 COSTS

5.8.1 Introduction. Experience in costs of protective construction is limited to a small number of actual construction projects and relatively few design studies. Thus three types of estimates may be made from data presently available.

a. Gross Facility Estimate. This provides a single unit cost ($ per sq. ft.) for an entire facility, including all structural, mechanical, electrical elements, access, utilities, etc. It can be used only in cases where the facility in question is typically the same as one for which actual construction cost experience is available. The limits of this experience are given in Para. 5.8.2.

b. Limited Cost Breakout. This reduces the total facility costs to nine elements, for each of which unit costs are provided as discussed in Para. 5.8.3. This method is applicable to cases where the level of protection, size and general configuration of the facility are known but advanced designs are not available and no direct experience data are available. It will be the most commonly used method in review of protective construction projects.

c. Detailed Cost Breakout. This requires a near-final design for the facility and consists of the procedures used conventionally in cost estimating. It will not be discussed further here. It is, of course, the

most reliable method. Special consideration must be given to the mechanical/electrical costs and to access requirements.

5.8.2 Gross Facility Estimate. Data on gross facility estimates are included in Vol. II. Cost data on fallout shelter construction for new facilities are presented in Table 5-4.

5.8.3 Limited Cost Breakout. Table 5-5 presents a list of the major elements which should be included in limited cost breakout estimates and suggested unit costs to be used in evaluating the contribution of each element. In some cases it is necessary to indicate ranges of unit costs and the estimator's judgment of the particular case will be required to select reasonable values within such ranges.

The costs of excavation and the structure itself have been shown as two separate items because depth of structure can vary over a wide range and excavation costs rise rapidly with depth of cover. The separation of the first three items of Table 5-5 requires that the estimator start with a minimum design concept. This concept must include the following items: type and size of basic structure, and depth of cover. From these he must determine at least approximate excavation and fill quantities and size of entrance structure. Figures 5-15 and 5-16 show the variation of cost of base structures as a function of type of structure. Figures 5-17, 5-18 and 5-19 show the variation for a particular structural type as a function of span or column spacing.

The additional costs of stairs or ramps in mounded or buried construction is to be included under Item 3 of Table 5-5. These items can be almost as costly as the basic structure, (Ref. 10), particularly when the basic structure is relatively small. Figures 5-15 and 5-16 are alike except that the curves of Fig. 5-16 include costs of entrance structure whereas those of Fig. 5-15 do not. For larger structures of a given type the cost of entrance structure should not increase appreciably with size of structure, any increase being primarily due to increase in depth of cover (if any) required by the larger basic structure. For example, a buried rectangular structure can be increased in size (plan area) without increase in depth of cover, and therefore without increase in cost of entrance structure. On the other hand, increase in the size of a buried dome requires an increase in the distance from the surface to the basic structure which must be provided for by the entrance structure. Thus the cost of entrance structure must be increased in this latter case. To obtain an estimate of entrance structure cost for a structure of different size than is represented by the curves of Figs. 5-15 and 5-16 the following approach is suggested.

a. By subtracting cost on the appropriate curve of Fig. 5-15 from the corresponding curve of Fig. 5-16 a cost-of-entrance-structure per square foot of the basic structure for which these curves are drawn is obtained

b. By multiplying the figure obtained in "a" by the area of the corresponding basic structure (listed in Fig. 5-16), a cost of entrance structure is found corresponding to the size of basic structure represented in Figs. 5-15 and 5-16.

c. The cost found in "b" must be multiplied by the ratio of entrance size for the structure of interest to the entrance size for the structure represented in Figs. 5-15 and 5-16.

Figure 5-20 gives approximate costs of protective doors, per square foot of opening, as a function of pressure and size. The cost curves also consider the attitude or orientation of the door in terms of whether it will be subjected to side-on or reflected pressure. Costs of stairs, ramps or other access are not included in Fig. 5-20.

5.8.4 Factors Affecting Costs. Some general discussion of cost factors follows:

a. Level of Protection. The strength and cost of structural components must increase with the overpressure level to be resisted, and the results of design studies of cost often have been presented in the form of cost factors vs design overpressure (Refs. 10, 11, 12). These cost factors may be dollars or dollars per square foot or ratios of cost at given overpressure level to cost of conventional (non-hardened) construction. When data given in this form are intended to reflect total costs (structural, mechanical-electrical including air conditioning where required, etc.) they must necessarily be more approximate and less reliable because total costs cannot be expressed as simple functions of overpressure. On the other hand, data of this kind covering only the structural costs, and for specifically defined structural types, can be sufficiently accurate to be useful if properly combined with estimated costs of the appropriate non-structural items. When dollars per square foot are presented as the cost factor the estimator should make certain that the areas used in computing such factors correspond to areas which are useful for the intended function. For example, in arch and dome construction perimeter areas may have to be discounted because of insufficient head room.

It is noted that overpressures are not always the governing factor in cost, from the point of view of protection level. In particular, for surface structures designed to low overpressure levels, protection against radiation hazards (in the form of minimum thicknesses of structural components and provision for air filtering) may be much more important than overpressure levels.

b. Size. From the point of view of direct structural costs, the required size, particularly the required clear span, is highly important. Figure 5-17 indicates the influence of clear span on cost for a simple rectangular form of structure. It must be emphasized that the question of size cannot very well be separated from the question of function. This may be illustrated by the fact that floor space provided in arched and domed structures is related to size of the structure and to whether multi-level floor systems can be utilized. If such utilization is feasible larger spans in structures of this kind may be attractive.

For certain special cases, such as aircraft shelters, the necessary spans, and particularly the correspondingly large exits and entrances, dominate the cost (Ref. 13).

c. Number of Personnel and Duration of Occupancy. Human occupancy adds much to the cost of protective construction. Utilities, messing facilities, food and water storage, air-conditioning, are costly items dependent upon the number of people who must live in the protected structure and the anticipated duration of their occupancy.

d. Function. It is apparent that all other listed factors are directly or indirectly related to the function of the protected installation. Both day-to-day and attack conditions of operation may be significant from the point of view of costs. The requirements for utilities, air conditioning, entrances, etc., may vary from a minimum in the case of a warehouse to a maximum in the case of a missile base or command center.

e. Geographical Area and Specific Site Location. Factors which influence the cost of conventional construction are at least equally significant to the cost of protected construction. Proximity to transportation, power and water, and the local availability of labor and materials are pertinent considerations. To a certain extent these can be accounted for by the application of "Location Factors" such as those tabulated in Ref. 14. Wherever possible, however, cost estimates should be based on designs which give full consideration to local conditions, and location factors should be used only when better information is not available.

The cost of protective construction may be very sensitive to conditions at the specific site. The type of soil to be handled is a major factor in costs of excavation and foundations; the importance of this factor increases with depth of construction. Ground water may add greatly to the costs of construction operations and entail additional expense for waterproofing the structure.

Whether a particular structure is isolated or part of a complex may influence direct construction costs as well as mechanical and electrical costs for the finished installation. Similarly the distances between structures in a complex may also influence direct and indirect construction costs.

f. Other. A number of factors other than those mentioned above can affect the costs of protective construction. These include the following:

(1) The degree of certainty in the design of the operational system which is to be protected

(2) The time urgency of construction

(3) Weather conditions at construction site.

Experience with construction of the Atlas and Titan missile bases has indicated that substantial increased costs can be incurred by change orders necessary as the basic missile system design is evolved.

To cover the above costs contingency items are included in budget estimates as well as allowance for government costs of engineering and design, supervision and inspection, and overhead. A factor of 21 percent of the basic

contractors bid price has been applied in the curves of this chapter. However, because of the uncertainties associated with construction of some protective facilities the factor may be as high as 30 percent.

5.8.5 References and Bibliography. Additional reference material on costs from a number of sources has been reviewed but specific reference to each report has not been indicated here. Some of the material is inconsistent because of the reasons noted above. All of the references consulted are listed in Para. 7.2, in Refs. 10 through 14, and in the Bibliography in Para. 5.8.6.

5.8.6 Limited Bibliography on Costs.

(1) "Report, Underground Installations", Guy B. Panero, for Chief of Engineers, under Contract W-49-129-Eng. 59,
"Summary"
"Working Conditions"
"Storage Depot"
"Chemical Process Plant"
"Precision Manufacturing Plant"
"Foreign Installations"
"Adaptations of Existing Mines for Emergency Use"
"Excavation and Construction Methods"
October 31, 1948, UNCLASSIFIED.

(2) "Classified Facilities High Point", Parsons, Brinkerhoff, Hall, and MacDonald, for Corps of Engineers, under Contract DA-49-080-Eng-2267, 24 June 1955, SECRET.

(3) "Underground Plants for Industry," January 1956. Department of Defense pamphlet prepared by the Office, Chief of Engineers, summarizing briefly the data contained in the reports listed under Item (1) above, UNCLASSIFIED.

(4) "Design of Underground Installations in Rock". Corps of Engineers Manual, EM 1110-345-431 (Draft), Chapter XI, Excavation Methods and Equipment, E. J. Longyear Company, under Contract 14-09-090-17 with Department of the Interior, Bureau of Mines, June 1955. UNCLASSIFIED.

(5) "Proceedings of the Symposium on Protective Construction", Office, Chief of Engineers, Sept. 21 and 22 1955. SECRET.

(6) "Preservation of Air Defense Capabilities, Design of a Fighter Air Base", Joint Air Defense Board, Colorado Springs, Colorado, 31 March 1953, SECRET, RESTRICTED DATA.

(7) "Project 51-VAR-NME Additional Facilities-Utility Study", (U) Parsons, Brinkerhoff, Hall, and MacDonald for Washington District, Corps of Engineers, December, 1956, SECRET.

(8) "Personnel Shelters and Protective Construction", (Draft), BuDocks TP-PL-8, November, 1958, UNCLASSIFIED.

(9) "Concept Study Report - March AFB Combat Operations Center", May 1959, with revised Appendix B, dated 11 June 1959. Donald R. Warren Company, under contract with the U. S. Army Engineer District, Los Angeles, Calif., (OUO)

(10) "March AFB Combat Operations Center, Basis of Design, Outline Specifications and Control Estimate", Dated December 1960, prepared by Donald R. Warren Company under contract with the U. S. Army Engineer District, Los Angeles, Calif., (OUO).

(11) "NORAD Site Investigations, Feasibility Report", dated January 1959, and "Report on Auxiliary Sites D and E", dated May, 1959, prepared by Parsons, Brinckerhoff, Hall and MacDonald, under contract with the U. S. Army Engineer District, Omaha, Nebraska, (SECRET).

(12) "SAGE Super Combat Center Site Adaptation Report, Deep Underground Facility (Hypothetical Site)", August 15, 1959, prepared by Praeger-Kavanagh-Waterbury under contract with the U. S. Army Engineer District, New York, (CONFIDENTIAL).

5.9 SHOCK ATTENUATION IN TUNNELS AND DUCTS

5.9.1 Entry. When a shock wave in the air encounters an opening such as a tunnel, shaft or duct leading to a buried facility, a shock wave is formed in the duct. The maximum peak overpressure in the duct occurs about 5 to 7 diameters inside the entrance, and its magnitude is a function of the peak overpressure in the wave outside and the angle between the centerline of the tunnel and the direction in which the shock is traveling.

Fig. 5-21 shows this relationship for three angles of incidence; face-on, oblique (45°) and side-on.

5.9.2 Attenuation

a. Straight Tunnel Sections. The decay in peak overpressure with distance as a shock wave is propagated down a straight tunnel is a function of the distance involved, the diameter of the tunnel, and the effective duration of the shock wave. The percent of the initial peak overpressure in the tunnel is plotted as a function of the length to diameter ratio and as a function of the effective duration of the shock wave in Fig. 5-22. The effective duration of the shock wave in the tunnel may be stated as a dimensionless parameter which is a function of the length of the tunnel, the maximum peak overpressure in the tunnel and the yield of the weapon employed. It may be computed from the following expression:

$$\tau = \left[\frac{t_{oo}}{L}\left(\frac{1\,ft}{millisec}\right) + \frac{1}{40}\left(\frac{p_{so}}{10\ psi}\right)^{1/2}\right] \tag{5-19}$$

where τ = dimensionless effective duration parameter of the shock wave

t_{oo} = 120 millisec. $\left(\frac{100\ psi}{p_{so}}\right)^{1.1} \left(\frac{W}{1MT}\right)^{1/3}$

L = length of tunnel involved

p_{so} = maximum peak overpressure in the tunnel (at entrance).

To use Fig. 5-22 enter the abscissa with appropriate values of L/D and $1/\tau$. For example; for L = 1000 ft., D = 10 ft., W = 1 MT and p_{so} = 100 psi; L/D = 100 and $1/\tau$ = 5. The ordinates for these two values are 85% and 61%, respectively. The product of the two (52%) is the percent of the initial peak overpressure which occurs at the distance 1000 ft down a straight tunnel section 10 ft. in diameter under the given conditions.

Although the maximum value of the peak overpressure occurs some distance inside the entrance, the total length of tunnel involved should be used to compute the pressure at distances greater than 10D. Attenuation of pressure in tunnels shorter than 10D in length may be neglected.

b. Effect of Tunnel Configurations. Just as bends and tees affect the pressure in a fluid flowing through a pipe, such configurations affect the peak overpressure in a shock wave traveling down a tunnel or duct. The effect of some of these configurations have been investigated by means of shock tubes and high explosives. The results of those empirical studies are summarized in Fig. 5-23. This figure is self explanatory. The decay in peak overpressure caused by a single 90° bend is very small and may be neglected.

5.9.3 Reflections

a. Long Tunnels. For tunnels greater than five diameters in length, the peak reflected overpressure on a door or other closure in the tunnel may be obtained from Fig. 5-24. That figure is a plot of the following expression.

$$p_r = 2p'_{so}\left(\frac{7p_o + 4p'_{so}}{7p_o + p'_{so}}\right) \tag{5-20}$$

where p_r = peak reflected overpressure, psi

p'_{so} = incident peak overpressure in the tunnel at the closure, psi

p_o = ambient atmospheric pressure, psi

b. Short Tunnels. Doors, blast valves or other closures are sometimes set back a short distance (< 5D) from the face of the mountain or

the junction of the tunnel with an entrance tunnel. In this short length of tunnel the shock picture is confused by reflections and/or vortices formed at the entrance. In the absence of empirical data or a theory which would permit a more accurate determination, it is suggested that the value of the peak reflected overpressure be assumed to vary linearly with the distance down the tunnel. For example, if a door were placed in a short tunnel at right angles to an entrance or bypass tunnel, the peak reflected overpressure on the door would be;

$$p_r' = p_{so} + \frac{x}{5D}(p_r - p_{so}); \; 0 \leq x \leq 5D \tag{5-21}$$

where p_r' = value of peak reflected overpressure on door, psi

p_{so} = side-on peak overpressure in entrance tunnel, psi

p_r = peak reflected overpressure as determined from Eq. (5-20)

x = length of stub tunnel.

Note that if the door were flush with the entrance tunnel the peak overpressure on the door would be the side-on overpressure in the entrance tunnel at that point.

5.9.4 Loading on Closures. The shape of the loading function on a closure in a tunnel may be represented in general by that shown in Fig. 5-25. The peak reflected overpressure may be obtained as indicated in Para. 5.9.3 above, and the duration of the spike of reflected pressure may be computed approximately from the following expression:

$$t_p = \frac{x}{U_r} + \frac{x}{a_r} \tag{5-22}$$

where t_p = duration of spike.

x = length of tunnel in front of door.

U_r = velocity of reflected shock wave with respect to the door.

a_r = velocity of rarefaction behind reflected shock front.

For use in Eq. (5-22), U_r and a_r are plotted together with the incident shock velocity as a function of the incident peak overpressure on the door in Fig. 5-26. These velocities should be used for relatively short tunnels only. If the tunnel leading to the closure is very long the velocity of the reflected wave will vary with the pressure as will the velocity of the rarefaction wave.

As indicated in Fig. 5-25, the spike of reflected pressure is superimposed on the pressure time function for the shock wave at the mouth of the tunnel leading to the closure. For example, for the case of a tunnel joined to an entrance tunnel the pressure time function would be that of the shock wave in the entrance tunnel at that point.

To compute the time required for a shock wave to enter a blast trap or debris pocket and return, the following expression may be used;

$$t_r = \frac{x}{U_s} + \frac{x}{U_r} \qquad (5\text{-}23)$$

where U_s = incident shock velocity.

Here again this expression should be modified for long tunnels to take into account the decay in pressure with distance. However, it will provide a reasonably accurate determination of the time required for traps up to 20D in length when large yield weapons are employed.

5.9.5 Limited Bibliography on Shock in Tunnels

(1) "First Information Summary of Blast Patterns in Tunnels and Chambers", Shock Tube Facility, Ballistic Research Laboratories, Aberdeen Proving Ground, Maryland, March 1960 (UNCLASSIFIED).

(2) Swatosh, J. J., Jr., and Birukoff, R., "Blast Effects of Tunnel Configurations, Final Test Report No. 17", Armour Research Foundation and Air Force Special Weapons Center, Albuquerque, New Mexico, AFSWC-TR-59-48, 1 October, 1959, (UNCLASSIFIED).

(3) Clark, R. O., and Coulter, G. A., "Attenuation of Air Shock Waves in Tunnels", BRL Memo Report No. 1278, DASA Report No. 1176, Ballistic Research Laboratories, Aberdeen Proving Ground, Maryland, June 1960 (UNCLASSIFIED).

(4) Clark, R. O., and Taylor, W. J., "Shock Pressures in Tunnels Oriented Face-on and Side-on to a Long Duration Blast Wave", BRL Memo Report No. 1280, Ballistic Research Laboratories, Aberdeen Proving Ground, Maryland, June 1960 (UNCLASSIFIED).

(5) Shear, R. E., and McCane, P., "Normally Reflected Shock Front Parameters", BRL Memo Report No. 1273, Ballistic Research Laboratories, Aberdeen Proving Ground, Maryland, May 1960 (UNCLASSIFIED).

(6) "Shock Wave Behavior in Tunnel-Adits Systems, Exploratory Phase", Misc. Paper No. 2-212, Waterways Experiment Station, Corps of Engineers, U. S. Army, Vicksburg, Mississippi, April 1957, (CONFIDENTIAL).

(7) Shapiro, A. H., "The Dynamics and Thermodynamics of Compressible Fluid Flow", The Ronald Press Company, New York, New York, 1958.

TABLE 5-1

CHANGE IN MAXIMUM VERTICAL DISPLACEMENT WITH DEPTH

Depth, ft.	Change in max. vert. displacement, reduction from surface values, in.	
	elastic component	permanent set component
	Coefficient of $\frac{p_{so}}{100\text{ psi}}\left(\frac{1000\text{fps}}{c}\right)^2$	Coefficient of $\frac{p_{so}-40}{30}\left(\frac{1000\text{fps}}{c}\right)^2$
25	0.6	0.25
50	1.2	0.50
75	1.8	0.75
100	2.4	1.0
125 or more	2.4	1.0

TABLE 5-2

ILLUSTRATIVE EXAMPLE - FREE FIELD
EARTH SHOCK EFFECTS AT VARIOUS DEPTHS

W = 8 MT
p_{so} = 200 psi
c = 2000 ft. per sec. down to a depth of at least h = 1000 ft.

Quantity	Direction	Max. Effects at Depth, ft.		
		0	50	100
Displacement, in.				
Elastic	Vert.	13.2	12.6	12.0
	Hor.	4.4	4.2	4.0
Plastic	Vert.	1.3	0.7	0
	Hor.	0.4	0.2	0
Total	Vert.	14.5	13.3	12.0
	Hor.	4.8	4.4	4.0
Velocity, ft. per sec.	Vert.	4.0	3.6	3.2
	Hor.	2.7	2.4	2.1
Accel., g	Vert.	150	18	8
	Hor.	150	18	8

TABLE 5-3

COMPARATIVE FEATURES OF POWER GENERATION EQUIPMENT

Type	Thermal Efficiency	Total Heat	Combustion Air	Fuel Oil Rate	Heat Rejection		
					To Stack	To Heat Sink	To Power Plant Area
Diesel Electric	35%	50.0×10^6 BTU/Hr.	1550 lbs/min. 21500 cfm	380 gph	14.0×10^6 BTU/Hr.	14.5×10^6 BTU/Hr.	4.0×10^6 BTU/Hr.
Open Gas Turbine 80% Re-generation	25%	68.0×10^6 BTU/Hr.	7020 lbs/min. 97500 cfm	470 gph	47.0×10^6 BTU/Hr.	2.0×10^6 BTU/Hr.	2.0×10^6 BTU/Hr.
Nuclear Steam Turbine	20%	65.0×10^6 BTU/Hr.	- - - - - -	- - - - - -	- - - - - -	65.0×10^6 BTU/Hr.	3.0×10^6 BTU/Hr.
Nuclear Gas Turbine	25%	68.0×10^6 BTU/Hr.	- - - - - -	- - - - - -	- - - - - -	49.0×10^6 BTU/Hr.	2.0×10^6 BTU/Hr.

NOTE: The tabulation is based on a plant capacity of 5000 KW (net electric).

TABLE 5-4

FALLOUT SHELTER CONSTRUCTION
NEW FACILITIES

Structure	Cost, Primary Structure for Normal Use	Primary Facility and Shelter Space Construction Description	Basement Shelter Area (Sq. Ft.)	Number* Persons Sheltered	Shelter** Cost	Shelter Cost** per Person Sheltered
Barracks 326-Man	$ 603,000	3-story masonry structure. Excavation of basement area under self-supporting concrete first floor	10,860	724	$ 47,000	$ 65
BOQ's 28-Man	238,000	2-story wood frame brick veneer structure, Replacement of self-supporting wood floor with concrete first floor and excavation of basement under	4,900	326	24,450	84
Reserve Center One-Unit	266,000	Masonry, part 1-story, part 2-story. Replacement of slab-on-grade first floor with self-supporting concrete floor and excavation of basement under	9,600	640	64,000	105
Typical Army Health Facility 75 Beds·plus clinics	2,520,000	Masonry, part 1-story, part 3-story. Excavation of basement under self-supporting concrete first floor	29,500 Plus 15,000 sq. ft. for treatment and evacuation center	1,970	128,000 65,000	65 Treatment & Evacuation Center
Typical Academic Facility	3,321,000	Masonry, part 1-, part 2-, and part 3-story. Excavation of basement under self-supporting concrete first floor	60,000	4,000	260,000	65

* Based on 15 sq. ft. gross per person.

** Includes 1 gal. water per person per day for 14 days, 1 toilet per 70 persons, and engine generator set for light and power. Does not include cost of collateral equipment.

TABLE 5-5

COST DATA - LIMITED COST BREAKOUT

Item	Costs (cy = cubic yard, SF = sq. ft., L.F. = Lin. ft.)			
	Above Ground	Mounded or Shallow u/g (25' Cover)	Deep Underground	Remarks
1. Excav. and Backfill	$/cy	$/cy	$/cy	
Backfill	2	3	-	
Excav. - Earth	3	3	6	
Excav. - Soft Rock	6	6	15	
Excav. - Hard Rock	10	10	25	
Excav. - Added Cost if Ground water problem	-	Add 50%	Add 100%	
2. Structural	$/SF	$/SF	$/SF	
Walls, roof, floors and foundations; bldg. frame and cladding exclusive of interior partitions, finishes and excavation.	9 (soft) See Figs. 5-15,5-17	See Figs. 5-15,5-17 5-18,5-19 5-20	Unlined 20 Lined 25	Structural costs vary with structural type, span, and design overpressure
3. Entrances, Doors	(See Figs. 5-15,5-16 and 5-20) Add 3 $/SF of door for radiation shielding at 25 psi	(See Figs. 5-15,5-16 and 5-20) Add for stairs or ramps	(See Figs. 5-15,5-16 and 5-20) Multiply by factor 1.5; access tunnels under Item 7	

TABLE 5-5 (Continued)

COST DATA - LIMITED COST BREAKOUT

Item	Costs (cy = cubic yard, SF = sq. ft., L.F. = Lin. ft.)			
	Above Ground	Mounded or Shallow u/g (25' Cover)	Deep Underground	Remarks
4. <u>Architectural</u> (Interior finishes partitions, etc.)	$/SF 4-9	$/SF 4-9	$/SF 10-22	Lower figures apply to simple interior layout, higher figures to multi-partitioned layout, acoustic treatment, etc.
5. <u>Mechanical</u> (Heating, ventilating, air conditioning, air filtration, plumbing)	$/SF 4-7	$/SF 4-7	$/SF 5-8 But add to excav. and struct. costs.	Includes 1.50 $/SF for decon: air filt., blast valves, etc. Refrigeration costs for cooling computers and other electronic equipment, and ice storage for emergency use must be added
6. <u>Electrical</u> (Lighting, electrical outlets, normal power connections)	$/SF 3-5	$/SF 3-5 But add to excav. and struct. space.	$/SF 4-6 But add to excav. and struct. costs.	Includes 1.0 $/SF for minimum administrative standby. For operational standby add 300 $/KW for generators and switchgear.

TABLE 5-5 (Continued)

COST DATA - LIMITED COST BREAKOUT

Item	Costs (cy = cubic yard, SF = sq. ft., L.F. = Lin. ft.) Above Ground	Mounded or Shallow u/g (25' Cover)	Deep Underground	Remarks
7. **Water Supply; Sanitary; ventilation shafts and tunnels; personnel shafts and tunnels**	250 $/person for water, 350 $/person for sewerage, 20D $/L.F. (for D > 10') 30D $/L.F. (for D < 10')	Same as for aboveground	350 $/person for water, 450 $/person for sewerage	For shafts and tunnels, cost of excav. or tunneling must be added.
8. Site Improvements Access roads, power transmission, communications tie-in, fencing, physical security features, heliports tone-down, radar countermeasures, etc.	Note: A breakdown of these items is not provided since the cost will vary widely for specific installations, depending upon the type of construction, geographical location, mission, existence of or proximity to other facilities and many other factors. However, the importance of considering the costs for such items during the initial planning stages cannot be overemphasized. On small projects they may amount to substantial increases in unit costs and greatly affect the validity of cost comparisons with other projects.			

TABLE 5-5 (Continued)

COST DATA - LIMITED COST BREAKOUT

	Item	Total Excavation Cu. Yards	Total Cost	Costs per Cubic Yard: Labor	Materials	Explo-sives	Power	Total
ed cost breakdown	Access Tunnels	33,337	$ 352,000	$ 6.82	$ 1.95	$ 1.30	$0.51	$10.56
avation in hard	Main Chambers	172,475	1,670,260	5.84	1.93	1.41	0.52	9.70
complished by	Cross Cuts and Misc.	32,922	506,659	10.31	2.86	1.68	0.54	15.39
3. Bureau of ring 1955-1958 ep underground tion	(subtotal)	238,734	2,528,919	6.61	2.07	1.39	0.53	10.60
	Rock Bolting	16,570(ea)	119,980	0.26	0.24	-	0.01	0.51
		(cost per bolt):		(3.60)	(3.42)	-	(0.21)	(7.24)
	Mining Equip.	--	371,813	-	1.56	-	-	1.56
	Engineering and Overhead	--	345,038	1.32	0.12	-	0.01	1.45
	Totals	238,734	$3,365,750	$8.19	$3.99	$1.39	$0.55	$14.12

exception of Item 9, all costs are referred to FY 1961 indices and Geographical Factor of
ts consist of contract costs plus about 20% for contingencies and inspection.

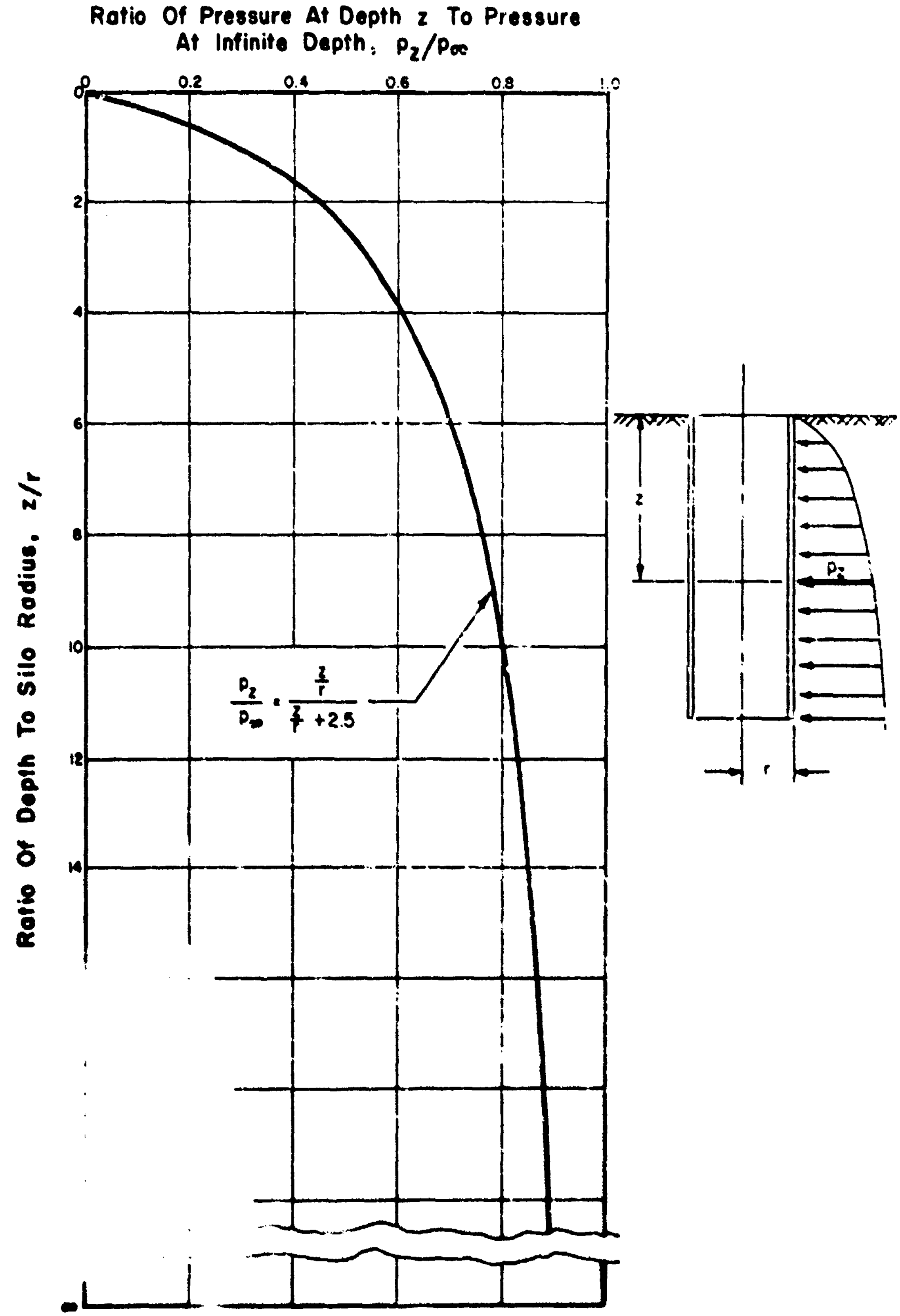


FIG. 5-1 HORIZONTAL DEAD LOAD PRESSURE DISTRIBUTION AGAINST SILO

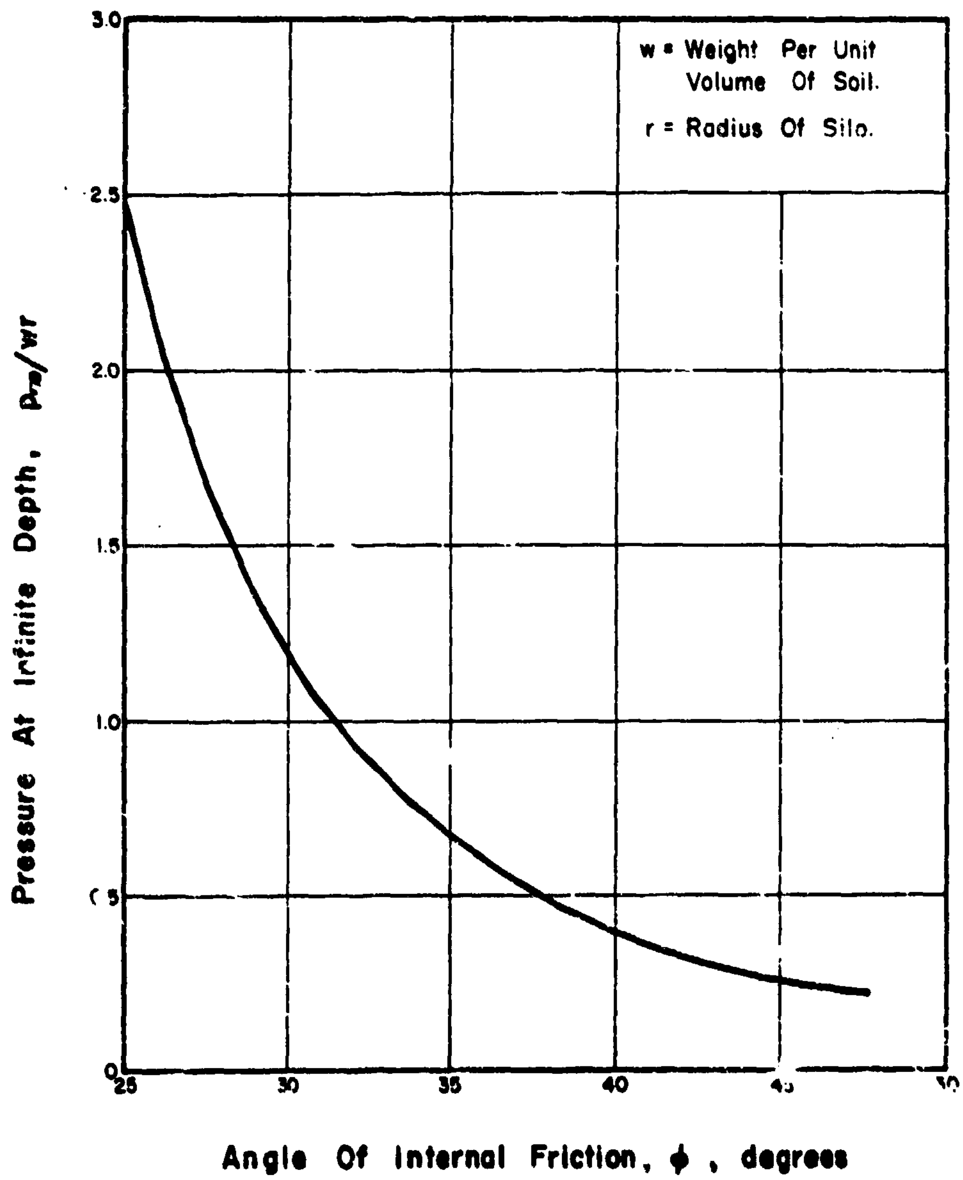


FIG. 5-2 MAGNITUDE OF HORIZONTAL DEAD LOAD PRESSURE ON SILO AT INFINITE DEPTH

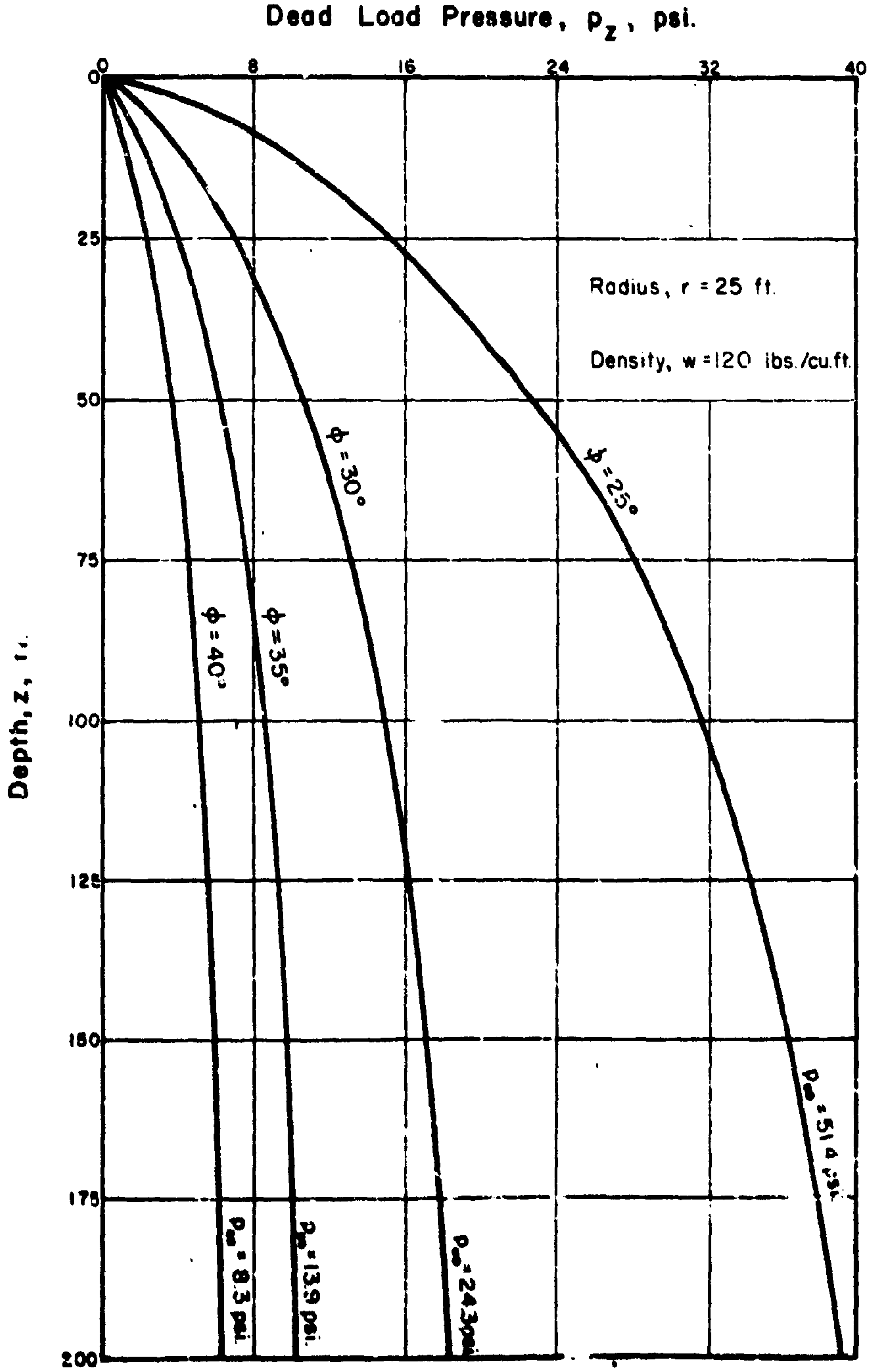
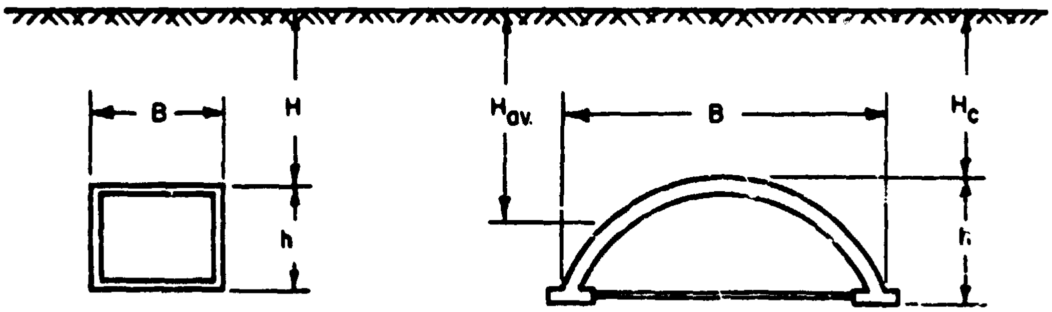


FIG. 5-3 HORIZONTAL DEAD LOAD PRESSURES ON 50-FOOT DIAMETER SILO FOR VARIOUS ANGLES OF INTERNAL FRICTION.



(a.) Box Section (b.) Arch

FIG. 5-4 NOMENCLATURE FOR TUNNELS

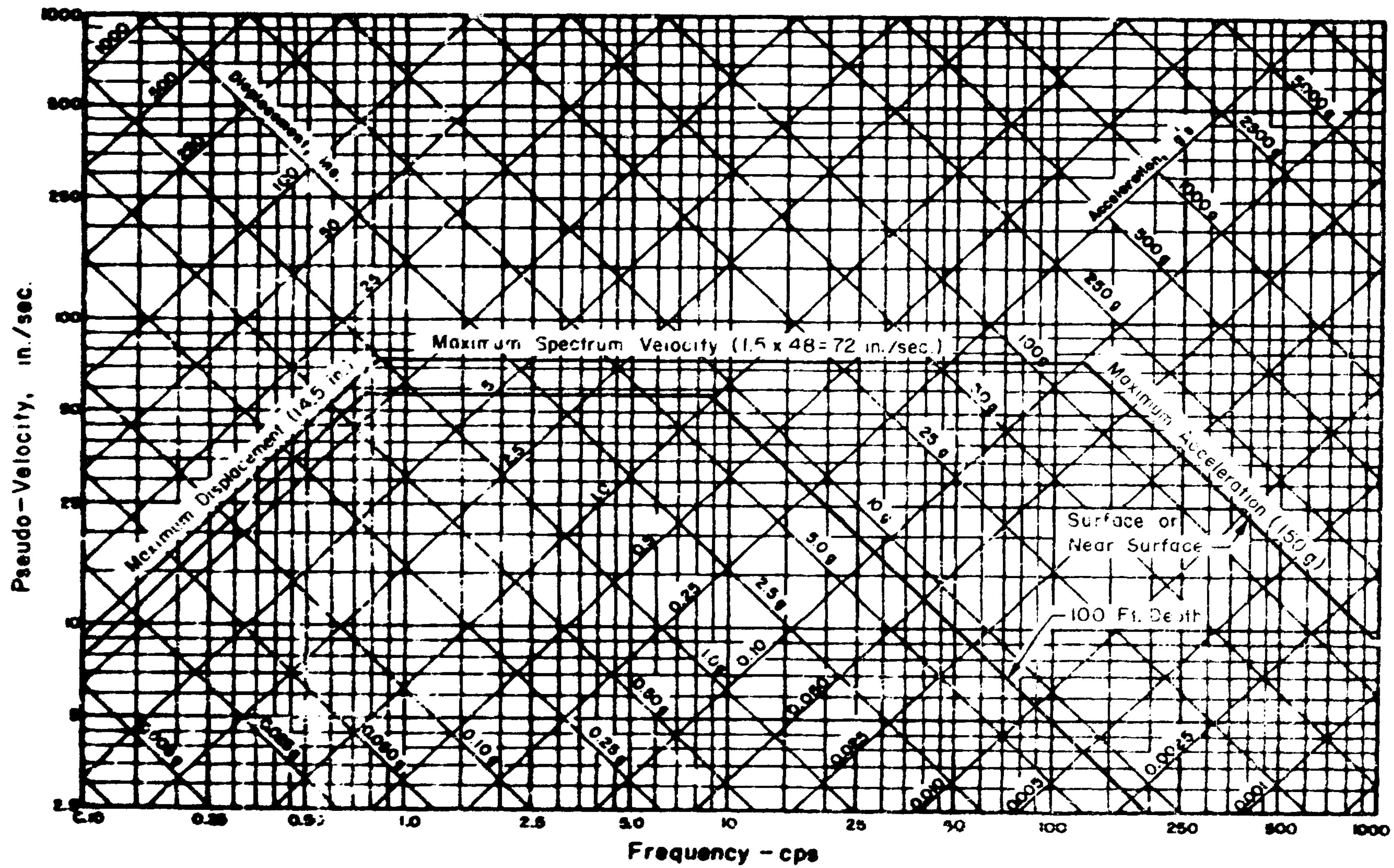


FIG. 5-5 SHOCK SPECTRUM BOUND FOR EARTH MOTION

NEWMARK, HANSEN & ASSOCIATES

OASD (P & I)

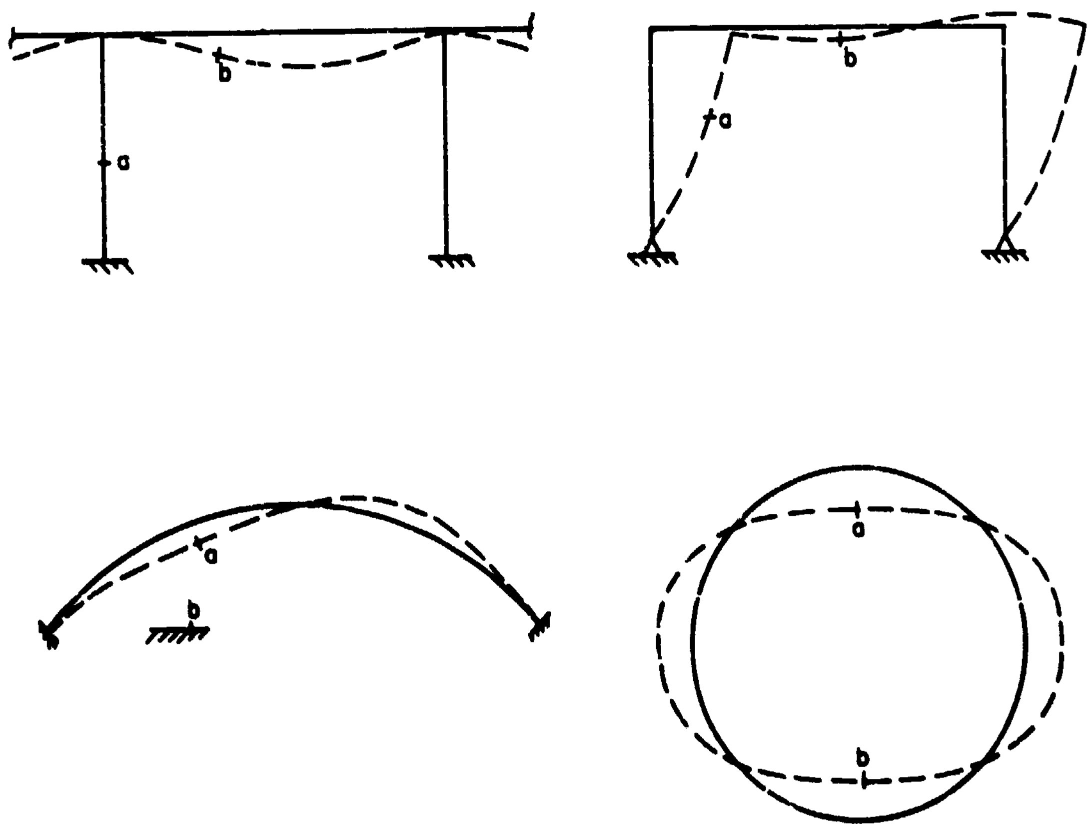


FIG. 5-6 RELATIVE DISPLACEMENTS WITHIN A STRUCTURE ASSOCIATED WITH STRUCTURAL DISTORTION.

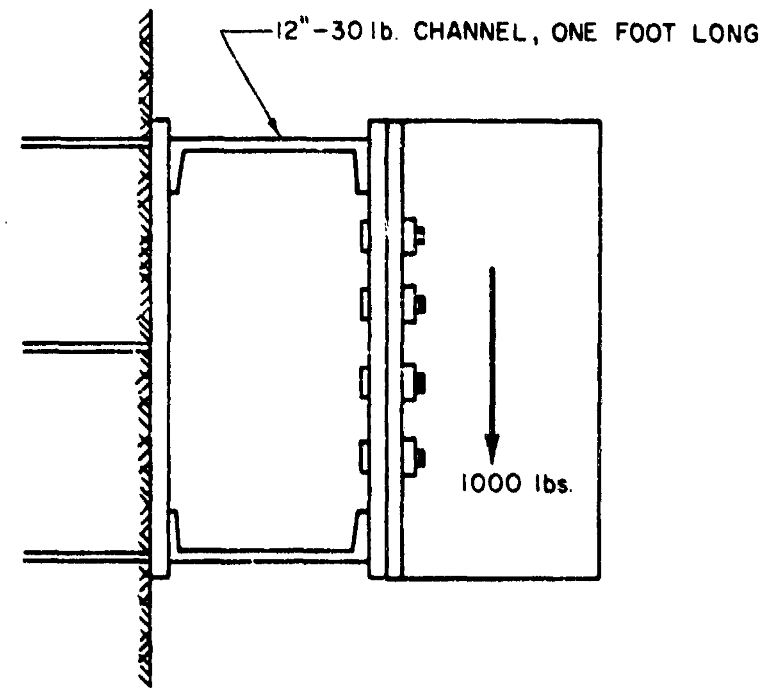


FIG. 5-7 EXAMPLE OF BRACKET MOUNTING

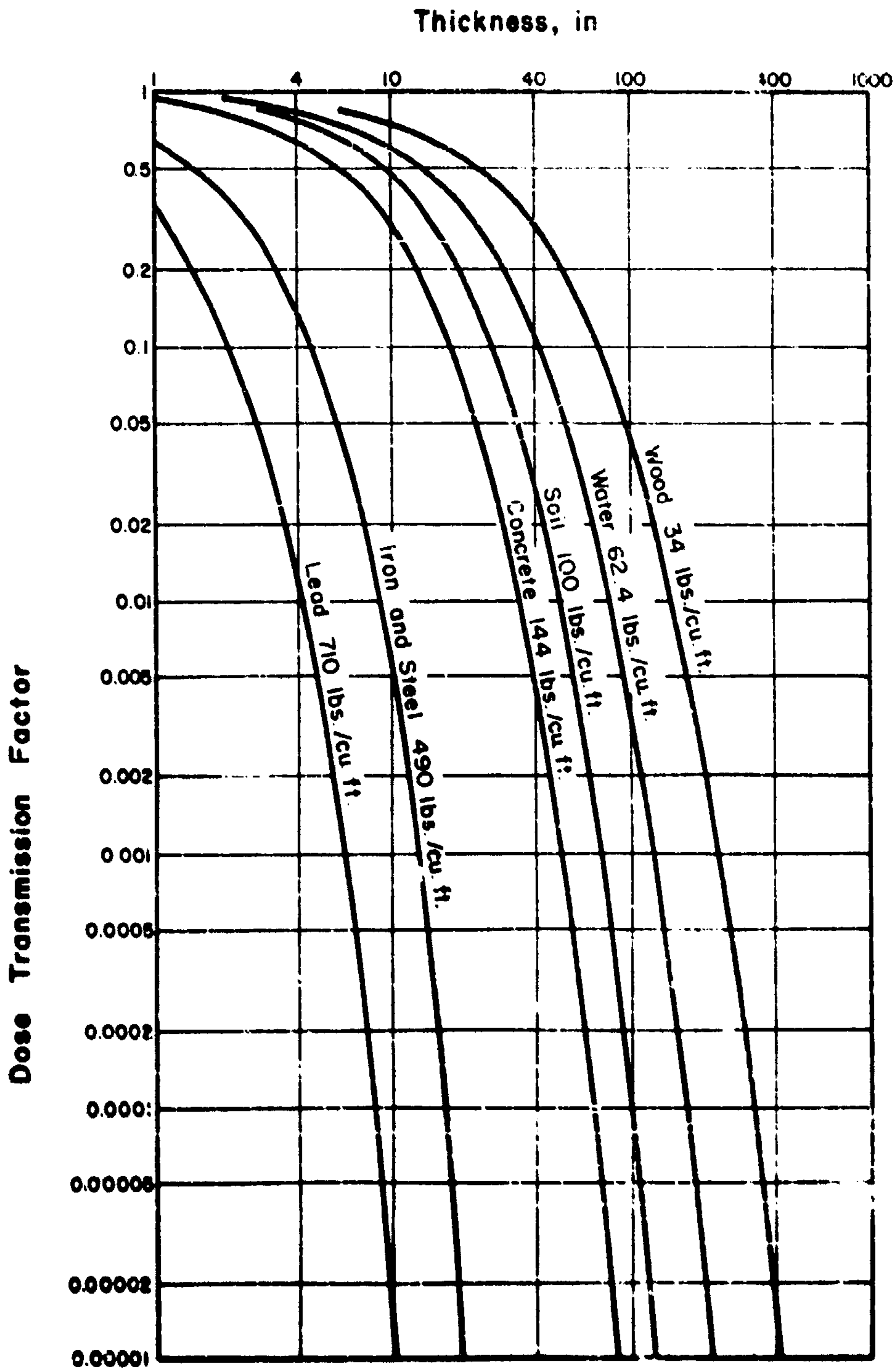


FIG. 5-8 SHIELDING FROM INITIAL GAMMA RADIATION

OASD (P & I)
SDS2 DEC '58-MAY '61

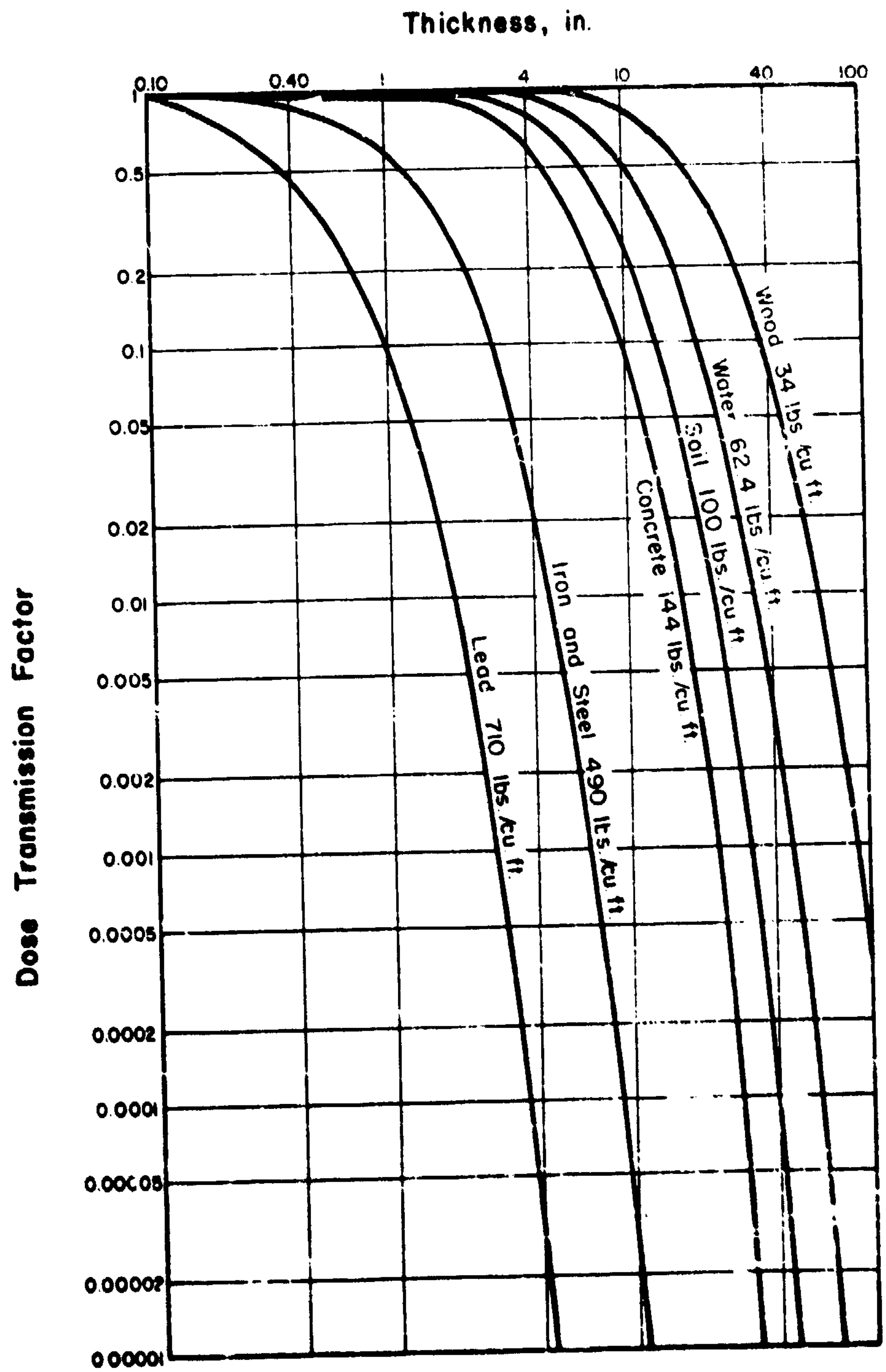


FIG. 5-9 SHIELDING FROM RESIDUAL GAMMA RADIATION

NEWMARK, HANSEN & ASSOCIATES

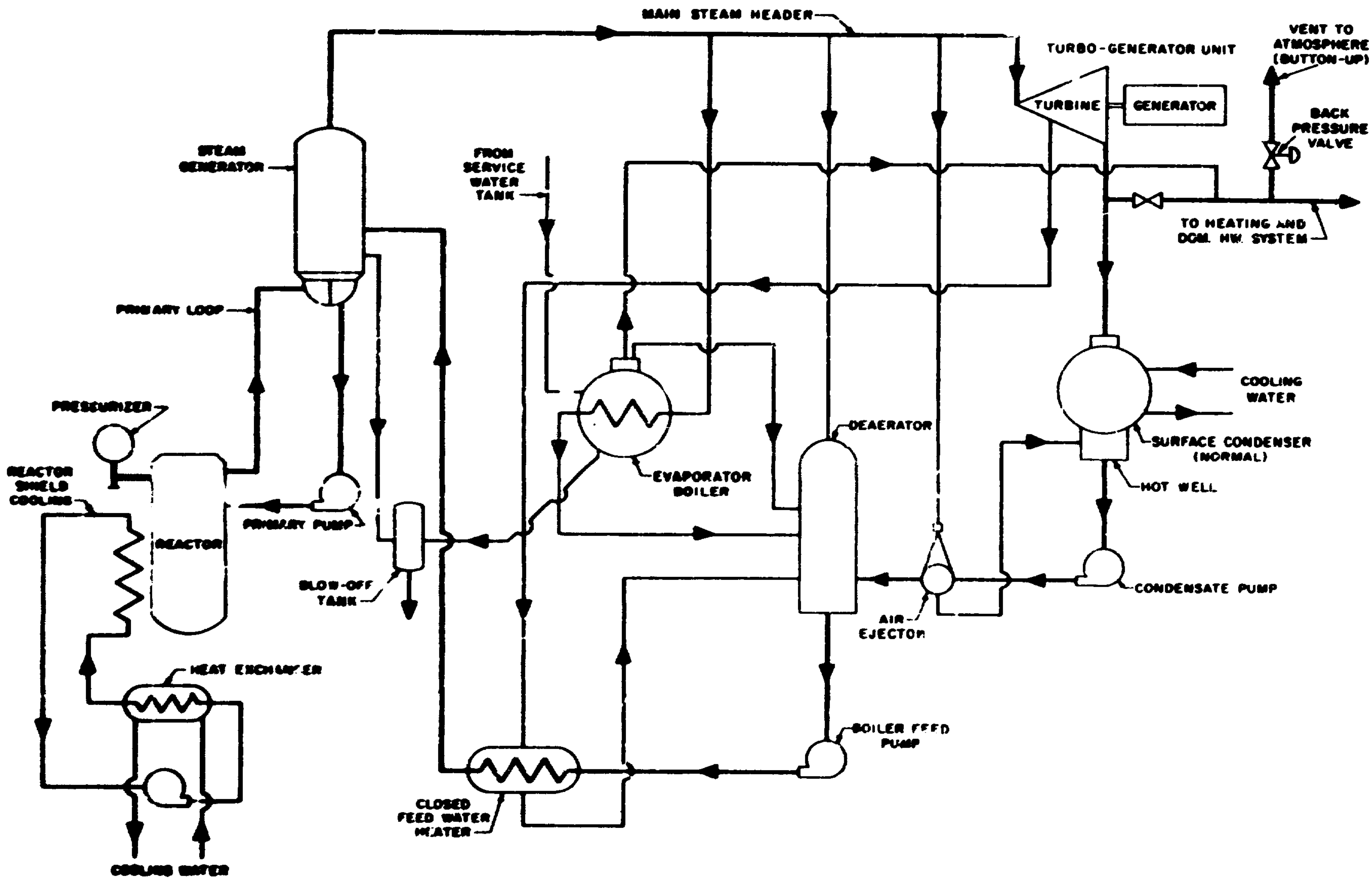


FIG. 5-10 CYCLE DIAGRAM FOR NUCLEAR-STEAM TURBINE POWER PLANT — PRESSURIZED WATER TYPE

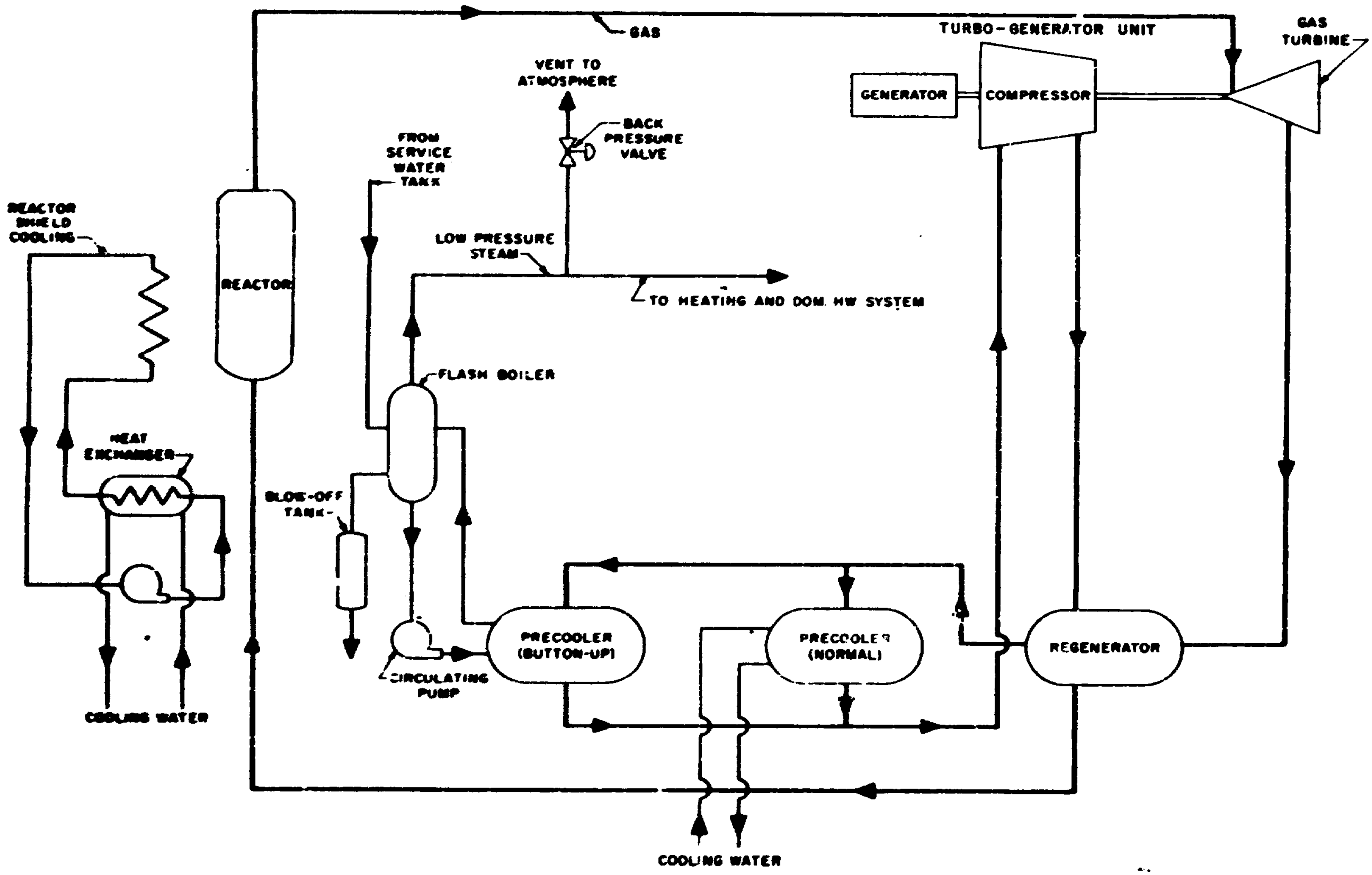
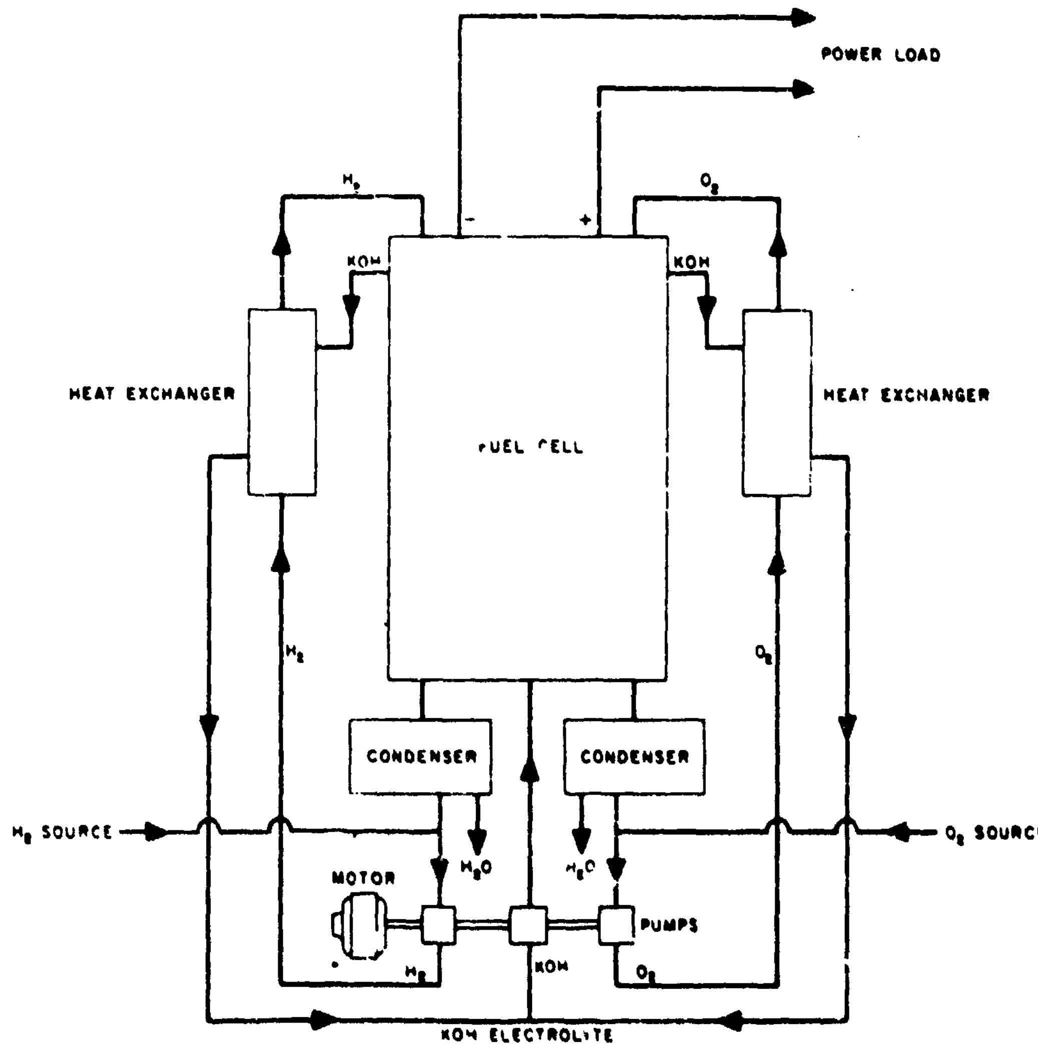
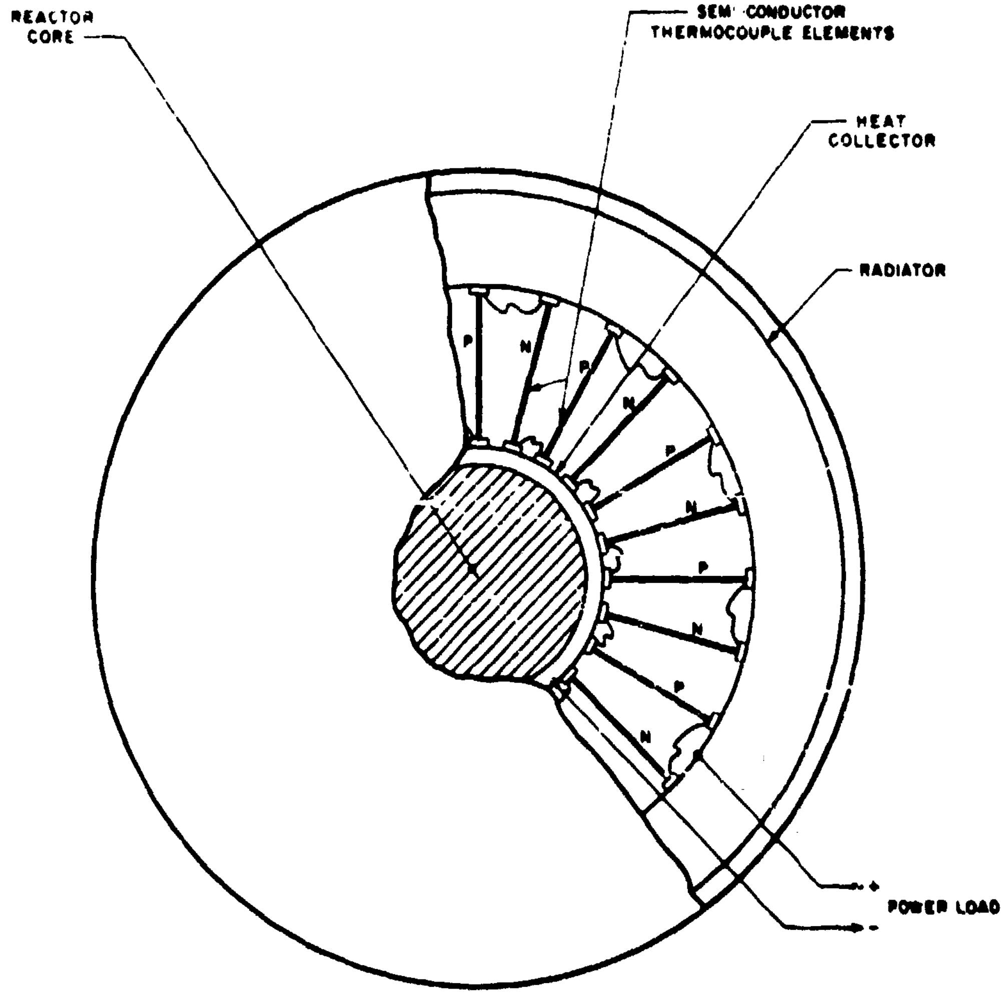


FIG. 5-11 CYCLE DIAGRAM FOR NUCLEAR-GAS TURBINE POWER PLANT



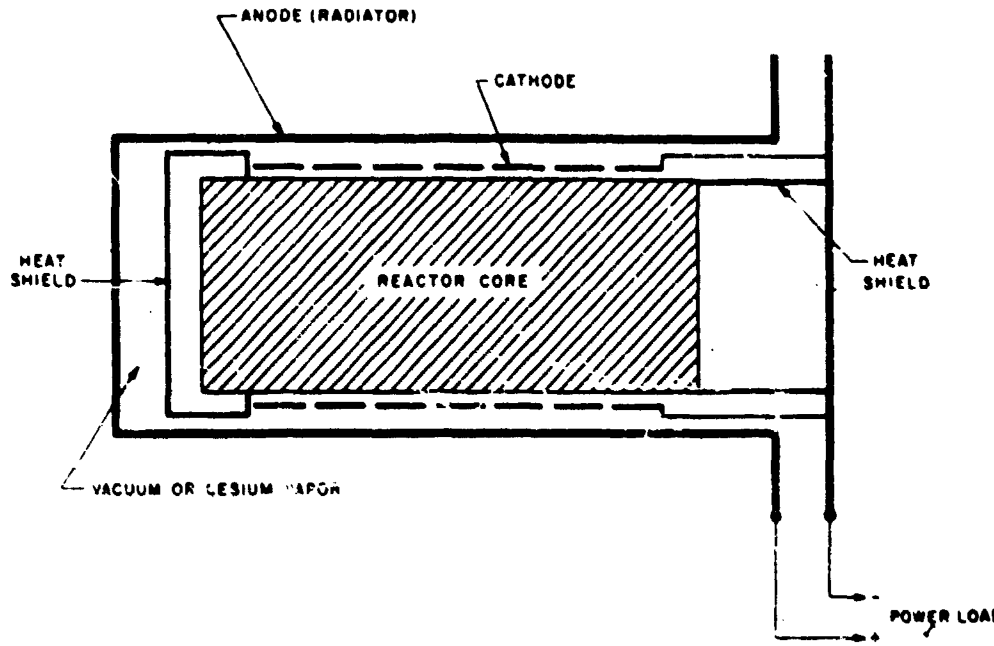
Hydrogen and oxygen gases enter the cell through specially-treated, carbon electrodes, and diffuse to the surface, where they come in contact with the electrolyte, a solution of potassium hydroxide. At the hydrogen electrode, the electrochemical reaction releases an electron, which flows through the external circuit and is accepted at the oxygen electrode. This flow of electrons is the current that powers electrical equipment. Ionic conductivity through the electrolyte completes the circuit, and the water formed in the reaction passes from the cell in the gas stream and is removed by suitable means.

FIG. 5-12 SCHEMATIC DIAGRAM FOR H_2/O_2 LOW TEMPERATURE FUEL CELL



Energy produced by the reactor is used to heat one junction of the two dissimilar semiconductor thermocouples. If a load is connected across the thermocouples an electric current will flow through this load.

FIG. 5-13 SCHEMATIC DIAGRAM FOR THERMO-ELECTRIC GENERATOR



Energy produced by the reactor is used to raise the temperature of the cathode to a point where electrons boil off the surface By placing a collector or anode in close proximity to the cathode, contained in a vacuum or cesium vapor environment, an electric current will flow through a closed external circuit

FIG. 5-14 SCHEMATIC DIAGRAM FOR THERMIONIC GENERATOR

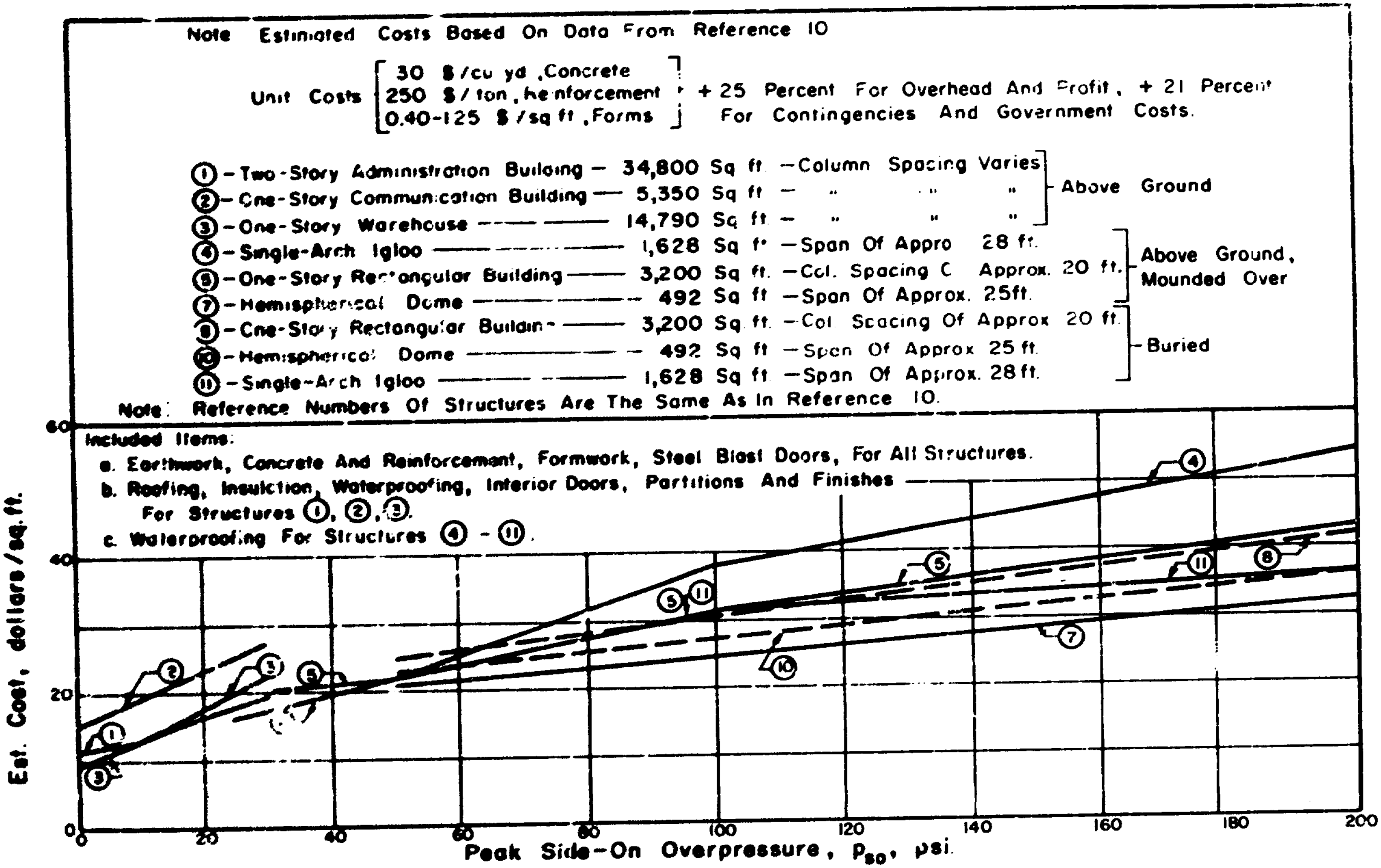


FIG. 5-15 ESTIMATED COST OF BARE STRUCTURE, EXCLUDING ENTRANCE STRUCTURES

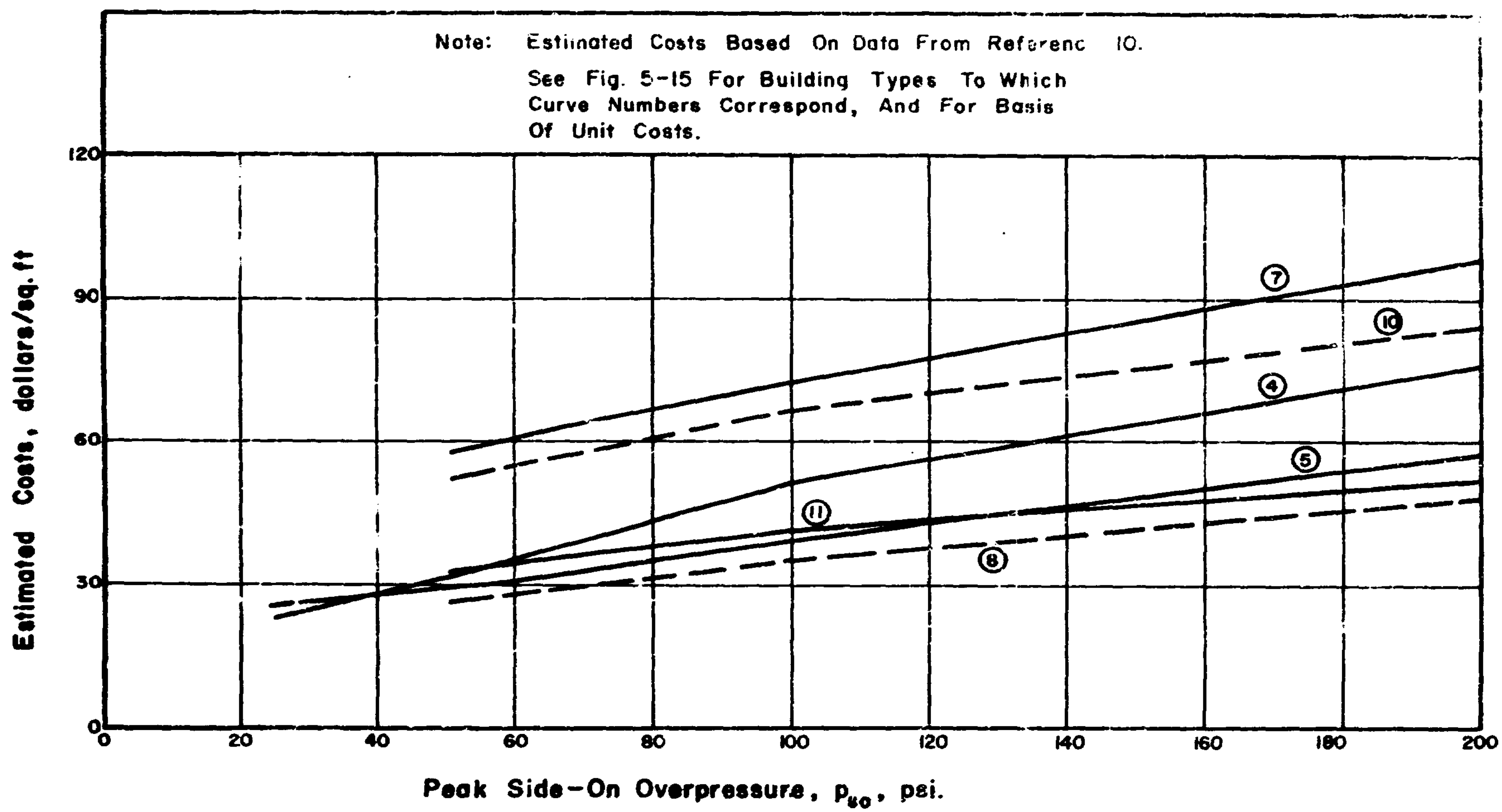


FIG. 5-16 ESTIMATED COST OF BARE STRUCTURE, INCLUDING SPECIAL ENTRANCE STRUCTURES.

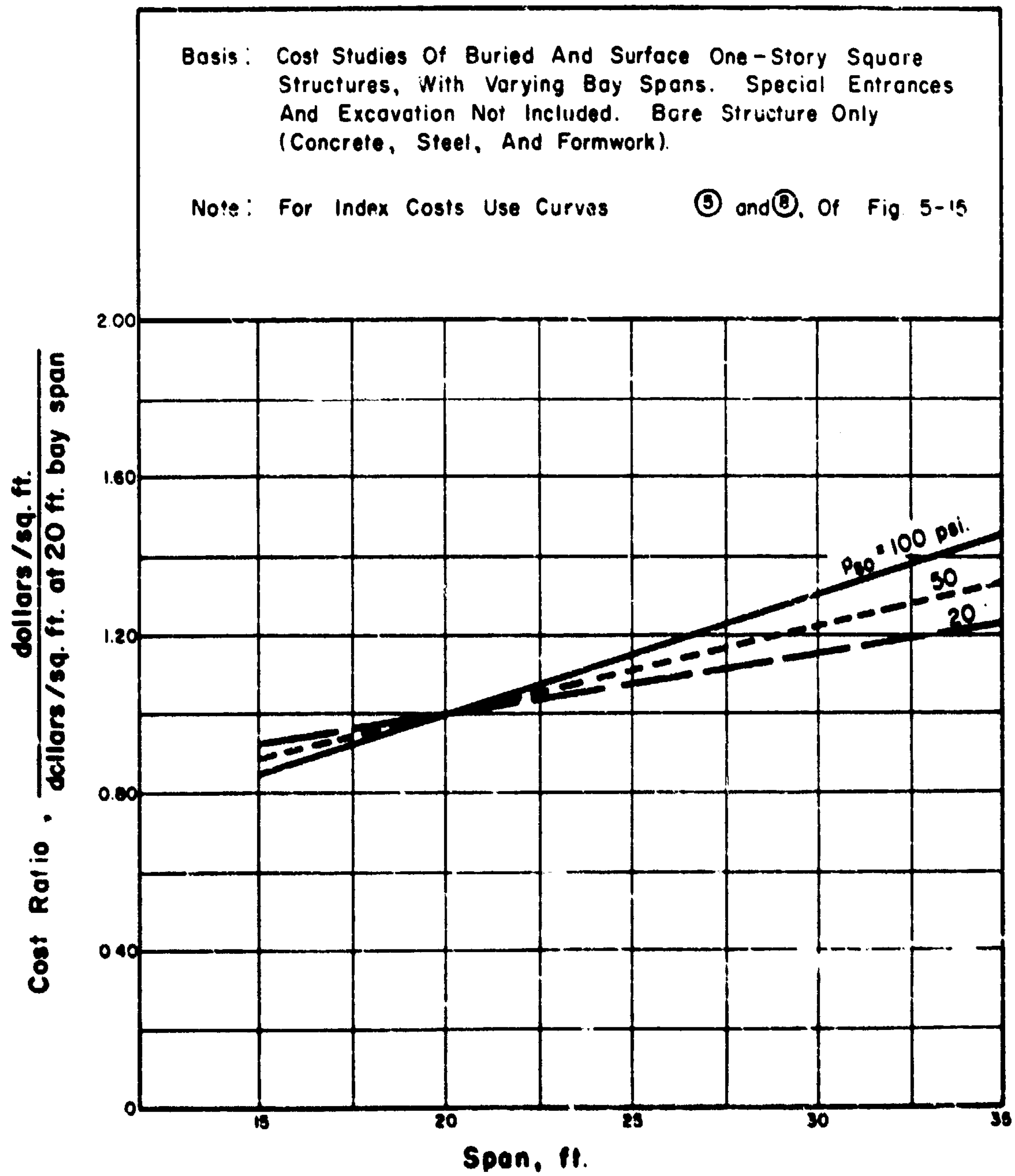


FIG 5-17 COST RATIO VERSUS SPAN FOR ONE-STORY RECTANGULAR STRUCTURES

Basis: Cost Studies For Buried, Hemispherical Domes. Special Entrances And Excavation Not Included. Bare Structure Only (Concrete, Steel, And Formwork).

Note: For Index Costs Use Curves 7 And 10 Of Fig. 5-15.

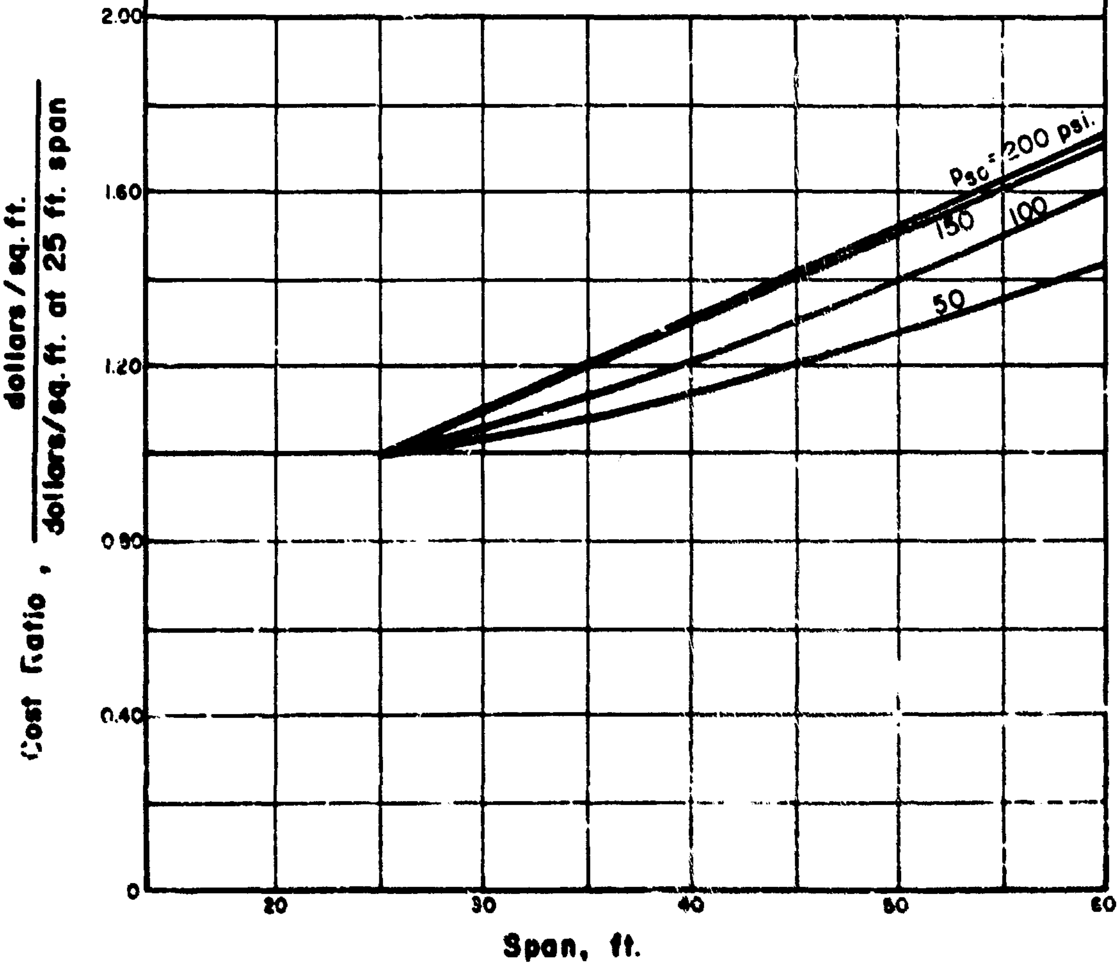


FIG. 5-18 COST RATIO VERSUS SPAN FOR DOME STRUCTURES

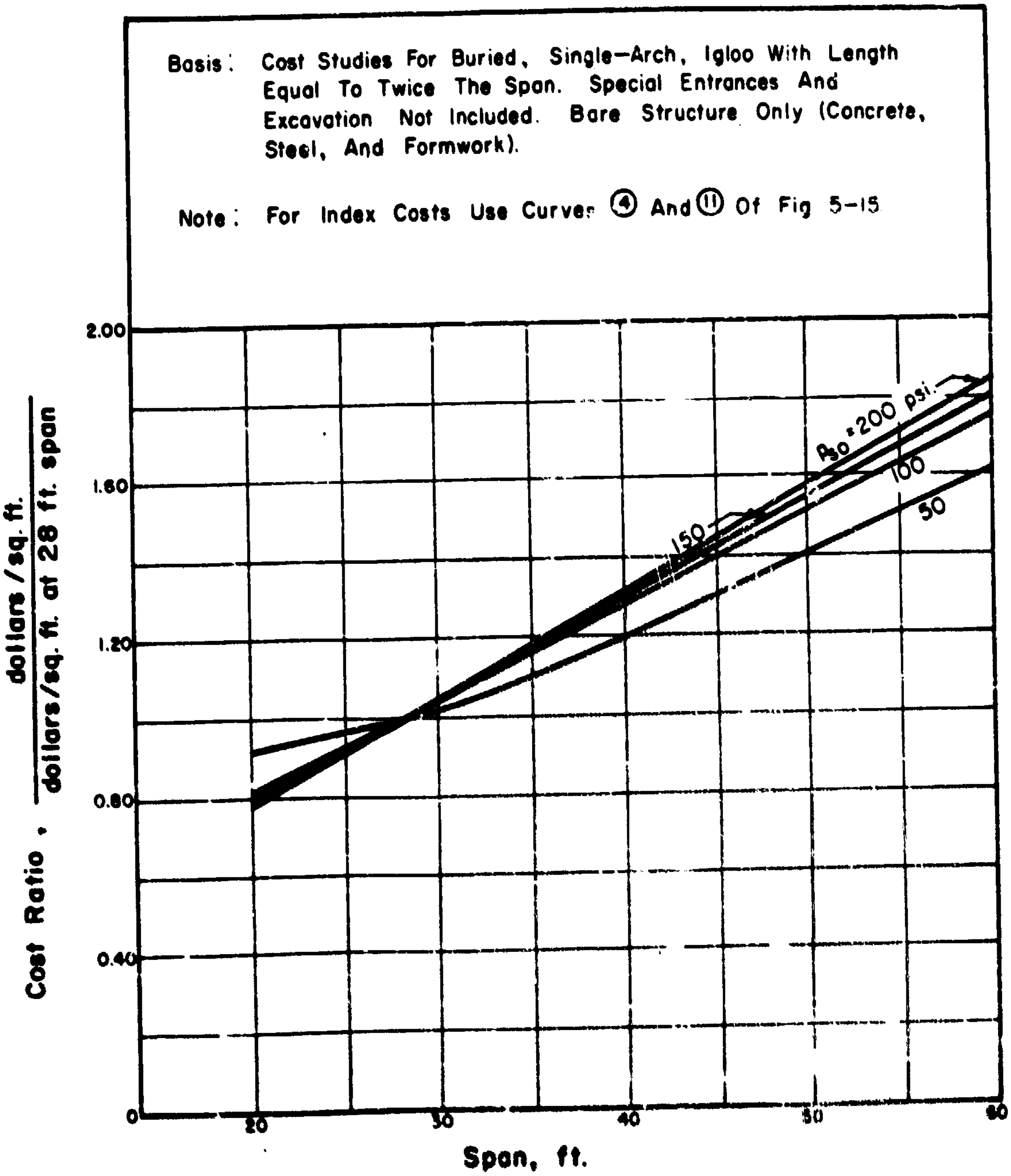


FIG. 5-19 COST RATIO VERSUS SPAN FOR ARCH (IGLOO) STRUCTURES

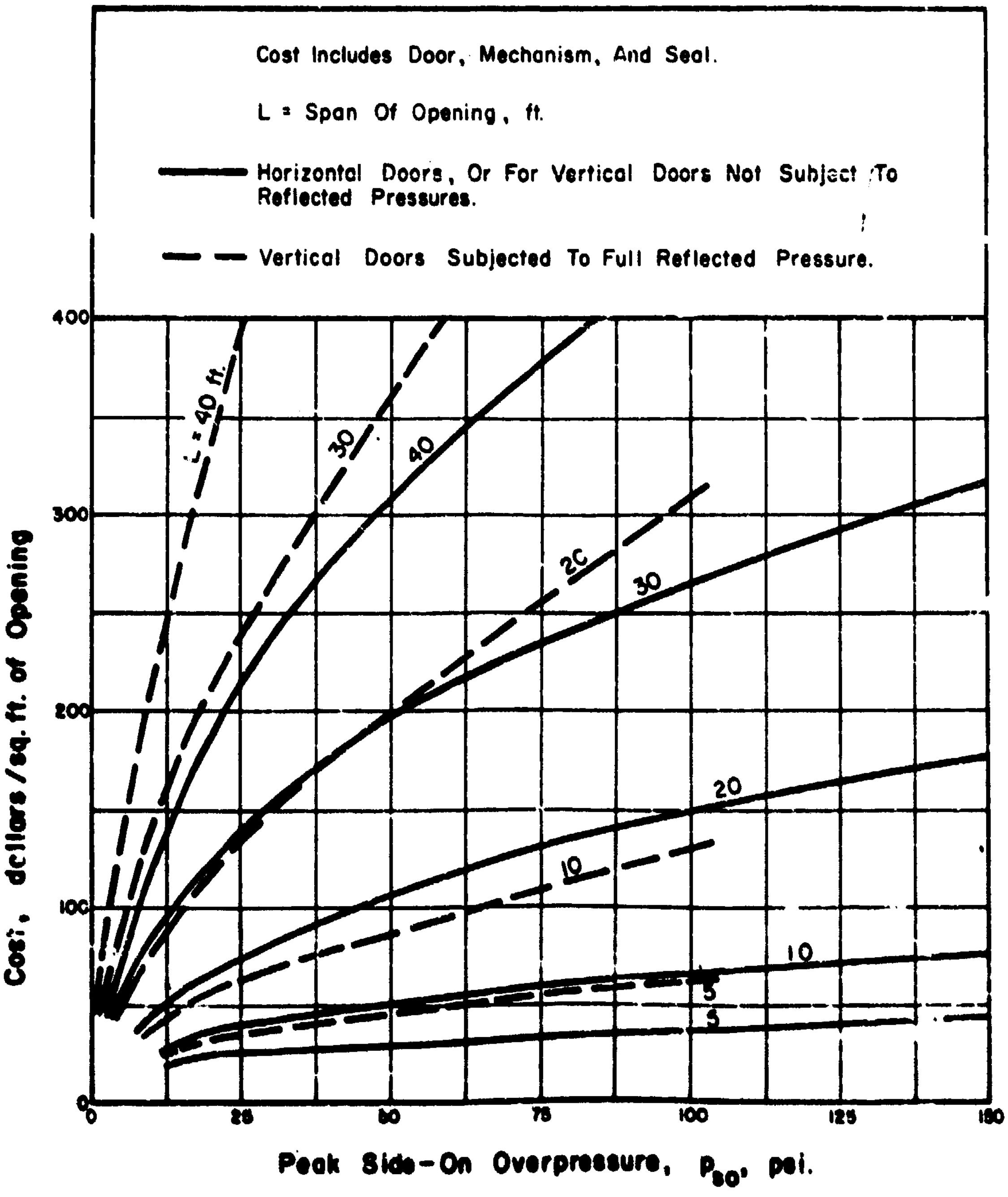


FIG. 5-20 COST OF PROTECTIVE DOORS

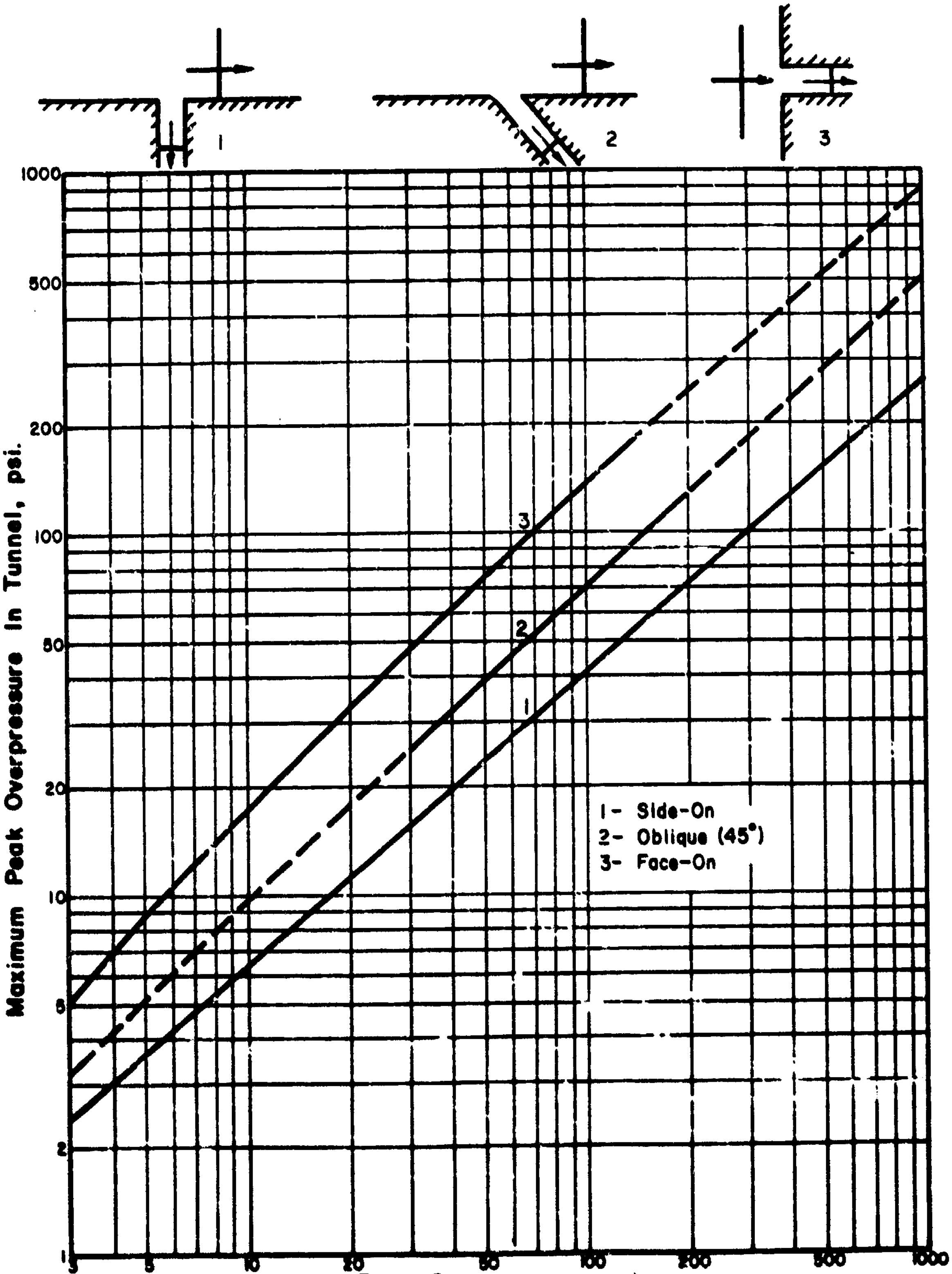


FIG. 5-21 MAXIMUM PEAK OVERPRESSURE IN A TUNNEL VERSUS INCIDENT PEAK OVERPRESSURE FOR THREE ANGLES OF INCIDENCE

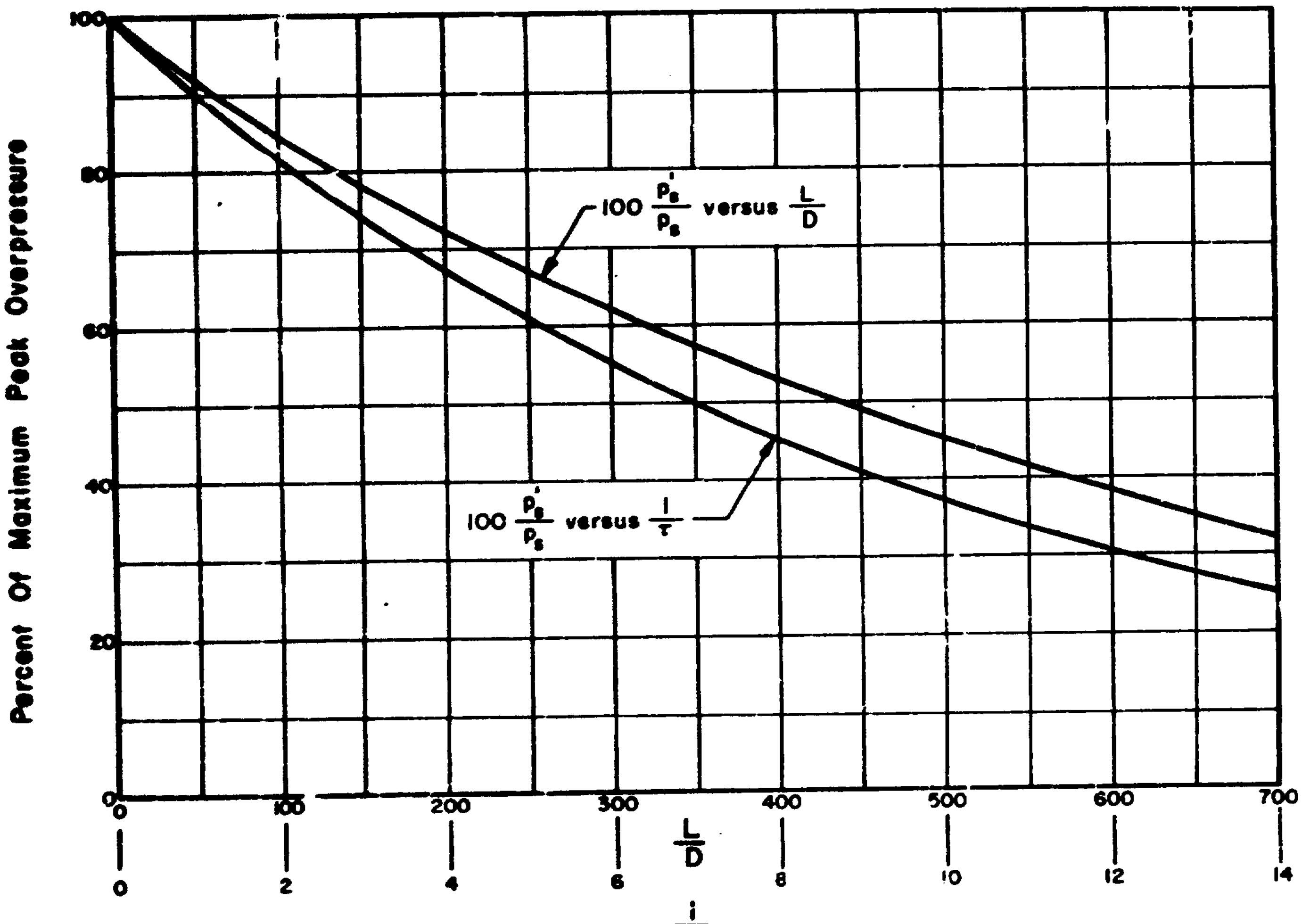


FIG. 5-22 PERCENT OF MAXIMUM PEAK OVERPRESSURE IN TUNNEL AS A FUNCTION OF $\frac{L}{D}$ RATIO AND EFFECTIVE SHOCK DURATION.

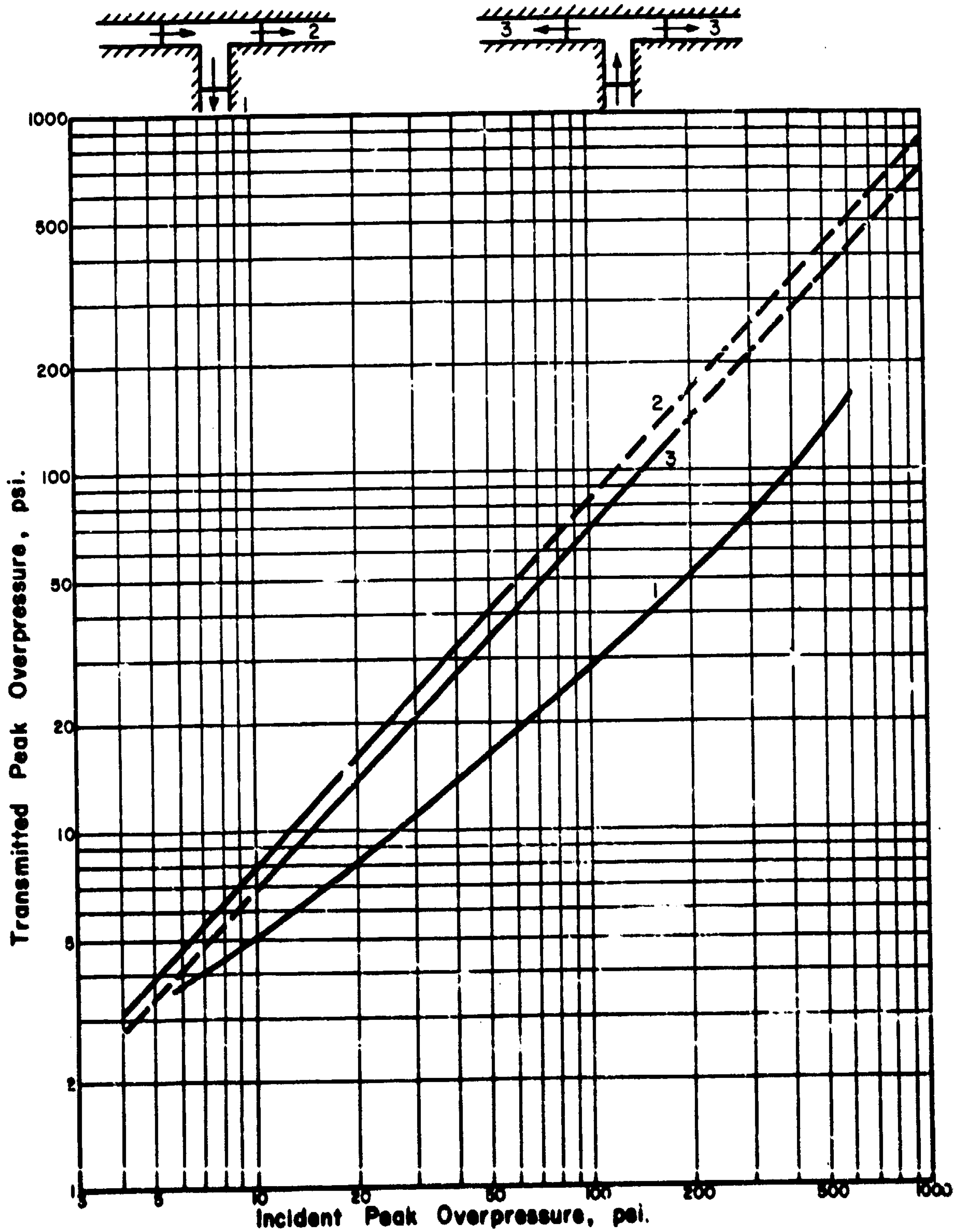


FIG. 5-23 TRANSMITTED VERSUS INCIDENT PEAK OVERPRESSURE FOR EQUAL AREA T-SHAPED TUNNEL JUNCTIONS

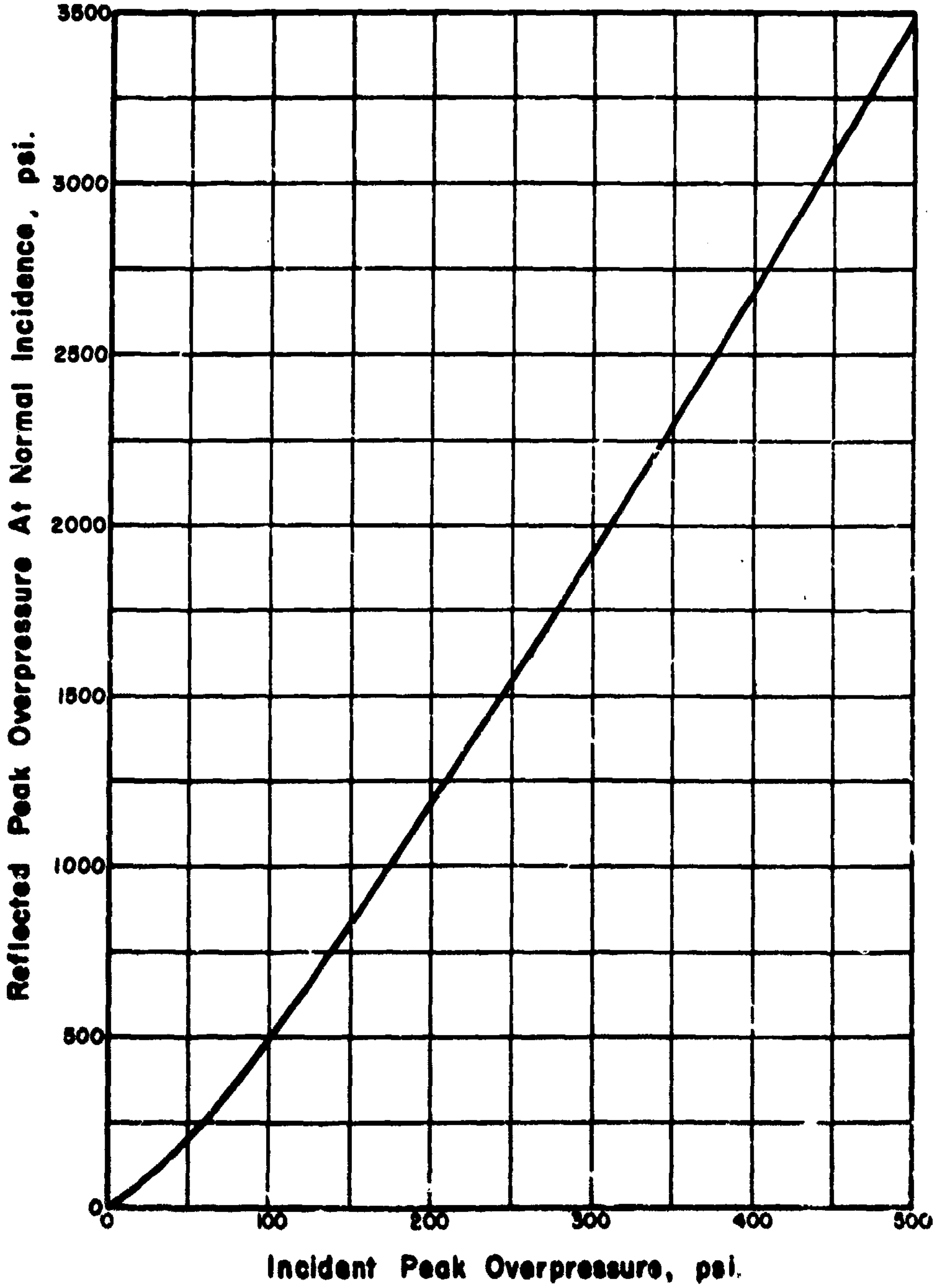


FIG. 5-24 REFLECTED PEAK VERSUS INCIDENT PEAK OVERPRESSURE AT NORMAL INCIDENCE

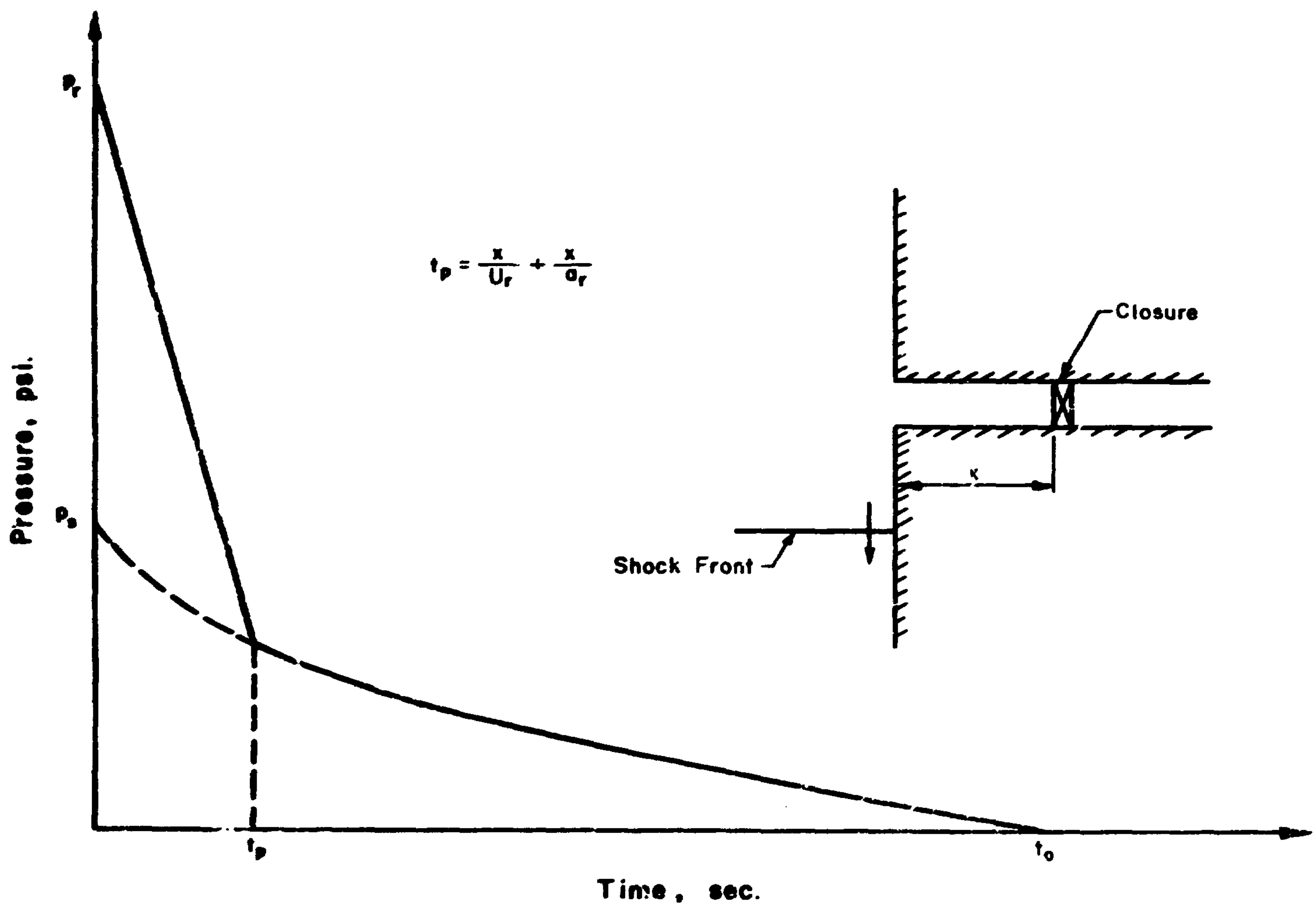


FIG. 5-25 SHAPE OF LOADING FUNCTION ON CLOSURE

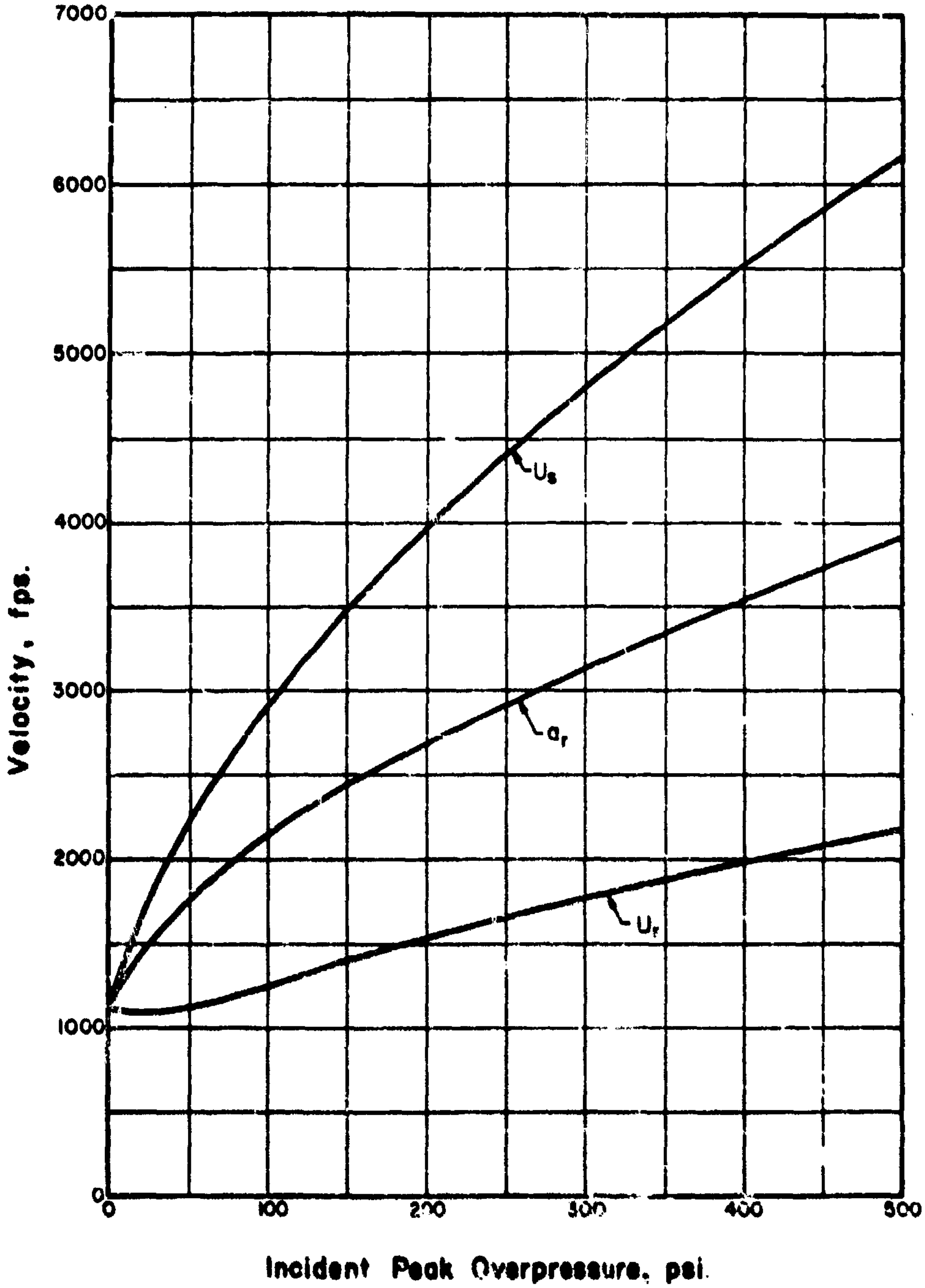


FIG. 5-26 VELOCITIES OF INCIDENT AND REFLECTED SHOCK WAVES AND OF RAREFACTION WAVE BEHIND REFLECTED SHOCK FRONT VERSUS INCIDENT PEAK OVERPRESSURE FOR SQUARE WAVE.

OASD (P&I) SD52 MAY '61

NEWMARK, HANSEN & ASSOCIATES

APPENDIX 5A. DESIGN CHARTS

5A.1 LIST OF DESIGN CHARTS

All design charts are contained as figures in this appendix. For convenience a list of the charts is given below. The use of the charts is explained and illustrated in Para. 5A.3. They are based upon the strength properties given in APPENDIX 5B and the general method of analysis presented in APPENDIX 5D. The latter appendix is also useful for special cases not covered by the charts. The design of tunnels and silos is based on the charts for arches given herein modified by the loading provisions contained in Para. 5.3.

Steel Beams (simply supported and continuous)

Flexural resistance	
Supporting one-way slabs, $\mu = 1.3$	Fig. 5A-4.1
Supporting one-way slabs, $\mu = 3.0$	Fig. 5A-4.2
Supporting square two-way slabs, $\mu = 1.3$	Fig. 5A-4.3
Supporting square two-way slabs, $\mu = 3.0$	Fig. 5A-4.4
(For non-square slabs use Fig. 5A-3.5)	
Shear resistance (supporting one-way and square two-way slabs)	
$\mu = 1.3$	Fig. 5[illegible]5
$\mu = 3.0$	Fig. 5A-4.6
(For non-square two-way slabs use Fig. 5A-3.9)	

Flat Slabs

Flexural resistance, $\mu = 1.3$	Fig. 5A-5.1
Flexural resistance, $\mu = 3.0$	Fig. 5A-5.2
Drop-panel factor for flexural resistance	Fig. 5A-5.3
Column capital factor for flexural resistance	Fig. 5A-5.4
Shear resistance, $\mu = 1.3$	Fig. 5A-5.5
Shear factors	Fig. 5A-5.6

Aboveground Rectangular Buildings

Resistance of one-story rigid frames, $\mu = 1.3$	Fig. 5A-6.1
Resistance of one-story rigid frames, $\mu = 3.0$	Fig. 5A-6.2
Resistance of one-story shear walls, $\mu = 1.3$	Fig. 5A-6.3
Resistance of one-story shear walls, $\mu = 3.0$	Fig. 5A-6.4

R/C Arches

Fully buried	
Required thickness for dead load	Fig. 5A-7.1
Required thickness for blast load, $\mu = 1.3$	Fig. 5A-7.2
Required thickness for blast load, $\mu = 3.0$	Fig. 5A-7.3
Partially buried	
Required thickness, $\mu = 1.3$	Fig. 5A-7.4
Required thickness, $\mu = 3.0$	Fig. 5A-7.5
Aboveground	
Required thickness, $\mu = 1.3$	Fig. 5A-7.6
Required thickness, $\mu = 3.0$	Fig. 5A-7.7

R/C Domes

Fully buried	
Use one-half the required thickness of arches having same span and depth of cover.	
Partially buried	
Required thickness, $\mu = 1.3$	Fig. 5A-8.1
Required thickness, $\mu = 3.0$	Fig. 5A-8.2

Aboveground

Required thickness, $\mu = 1.3$ Fig. 5A-8.3

Required thickness, $\mu = 3.0$ Fig. 5A-8.4

Steel Arches

Fully buried

Required area, $\mu = 1.3$ Fig. 5A-9.1

Required area, $\mu = 3.0$ Fig. 5A-9.2

Partially buried

Required area, $\mu = 1.3$ Fig. 5A-9.3

Required area, $\mu = 3.0$ Fig. 5A-9.4

Aboveground

Required area, $\mu = 1.3$ Fig. 5A-9.5

Required area, $\mu = 3.0$ Fig. 5A-9.6

Columns

Strength of R/C columns under axial loads Fig. 5A-10.1

Strength of R/C beam-columns Fig. 5A-10.2

Strength of steel columns Fig. 5A-10.3

(For strength of steel beam-columns see APPENDIX 5B)

Footings

Resistance of square column footings, $a/L = 0.1$ Fig. 5A-11.1

Resistance of square column footings, $a/L = 0.25$ Fig. 5A-11.2

Flexural resistance of wall footings Fig. 5A-11.3

Shear resistance of wall footings Fig. 5A-11.4

5A.2 NOTATION USED IN CHARTS

a = beam or column width

A = cross-section area of steel arch

b = spacing of beams, rigid frames or shear walls

b_f = width of steel column flange

B = span of arch or dome

c = width of column capital in flat slabs

d = depth to steel in concrete beams and slabs

d_p = effective depth of drop panel

d_z = plastic bending modulus divided by area of steel arch cross section

D = total thickness of dome or arch

f'_c = 28-day compressive strength of concrete

f'_{dc} = dynamic compressive strength of concrete

f_{dy} = dynamic tensile yield stress of steel

f_{yv} = dynamic shear yield stress of steel
h = height of building frame
H_{av} = average depth of earth cover
K = column capital factor for flat slabs
K_1 = shear factor for flat slabs
K_2 = shear factor for flat slabs
ℓ = projection of wall footing
L = span length
m = wide dimension of concrete column section
ΣM = sum of moment capacities at column ends in a frame
n = narrow dimension of concrete column section
p = width of drop panel in flat slabs
p_m = peak blast pressure
p_{mf} = flexural resistance of flat slabs
p_{mv} = shear resistance of flat slabs
p_{so} = peak side-on blast pressure
P_u = ultimate column strength
r = radius of dome or arch
t = thickness of shear wall
T_n = natural period
t_w = web thickness of steel beam
w = weight of steel column, lb. per ft.
W_s = bearing pressure under footings
X = drop panel factor for flat slabs
X' = drop panel factor for flat slabs
Y = width of building frame
Z = section modulus of steel beam
α = ratio of short to long sides of two-way slab
β = one-half internal angle of arch or dome
γ = factor for long beam under two-way slab
η = shear factor for beams under two-way slabs
λ_f = flexural reinforcement factor for diagonal tension strength of concrete slabs and beams
λ_s = pure shear factor for slabs and beams with inclined reinforcement
λ_v = web reinforcement factor in concrete beams
μ = ductility ratio
φ_c = steel percentage at mid-span of concrete beams
φ_e = effective steel percentage at support of concrete beams
φ_T = total steel percentage (both faces)
φ_v = percentage of web reinforcement (vertical stirrups)
φ'_v = percentage of inclined web steel

Ω = two-way slab factor for flexural resistance

5A.3 METHODS OF USE OF CHARTS

5A.3.1 <u>One-way Slabs</u>. Charts for continuous and simply-supported one-way slabs are given herein. Figures 5A-1.1 and 5A-1.2 provide the required thickness for flexure in terms of the peak blast pressure p_m. The percentage of positive steel at midspan φ_c, and the effective negative steel at supports φ_e, must be known or selected. The latter would be taken as zero for simply-supported spans. Where a slab frames into a wall or is continuous with other panels of slab, or both, the resisting moment that can be developed at the support is the lesser of the following two quantities:

(1) The resisting moment that can be developed in the slab itself, which is measured by the actual value of the end reinforcement $\bar{\varphi}_e$.

(2) The resisting moment that can be developed by the members restraining the slab, when these members act in a manner consistent with the over-all loading applied.

The effective steel percentage φ_e is computed as the amount of steel that can develop the smaller resisting moment of the two values described above. If item (1) governs, $\varphi_e = \bar{\varphi}_e$. But if item (2) governs, φ_e will be less than $\bar{\varphi}_e$, and will in general bear the same ratio to $\bar{\varphi}_e$ as the moment in item (2) bears to the moment in item (1). This definition is consistent with a zero value of φ_e at a simple support.

Figure 5A-1.3 gives the required thickness of a one-way slab to prevent a failure in pure shear if the slab does not have inclined web steel in the region of high shear. The effect of inclined web reinforcement on pure shear strength is given in Fig. 5A-1.4. Figures 5A-1.5 through 5A-1.8 give the required slab thickness if diagonal tension controls. If web reinforcement is required, a percentage of at least 0.25 percent should be used to insure ductility.[1]

Charts for pure shear are given only for $\mu = 1.3$ because this failure mode has little ductility, and design for higher ductility ratios is not recommended. It is permissible, however, to design for $\mu = 1.3$ in pure shear and at the same time use $\mu = 3.0$ for flexure and diagonal tension. If there is no web reinforcement, μ in diagonal tension should also be limited to 1.3.

Under some conditions, the criteria for pure shear and diagonal tension strength which form the basis of Figs. 5A-1.3 through 5A-1.8 are invalid and flexural strength controls. The limits of applicability of the pure shear and diagonal tension charts are given in Figs. 5A-1.9 and 5A-1.10.

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[1] The first edition of this Review Guide contained additional charts for the "cracking strength" of beams without web reinforcement. More recent investigations have indicated that this is an unnecessary complication.

As an example consider a continuous roof slab with a 12-foot span for an unburied building which is to be designed for a peak overpressure (p_m) of 100 psi. Assuming that deflection is not a critical factor, design for a ductility ratio (μ) of 3.0 in flexure.

Given:

p_m = 100 psi	$\mu = 3.0$
f_{dy} = 52,000 psi	(see APPENDIX 5B)
L = 12 ft.	Select $\varphi_c = \varphi_e = 1.5\%$
No web reinforcement	$f'_c = 3000$ psi, $f'_{dc} = 3750$ psi

Flexure, Fig. 5A-1.2

$$(\varphi_c + \varphi_e)\, f_{dy} = (1.5 + 1.5)\ 52{,}000$$

$$= 156{,}000 \text{ psi}$$

Read, $d/L = 0.10$

Pure Shear, Fig. 5A-1.3

$p'_m = p_m$ since φ_e is same at both ends

Read, $d/L = 0.11$

In this case, the depth required for pure shear is only slightly in excess of that required for flexure; consequently, to reinforce for pure shear is impractical. However, for very short spans under high pressures, pure shear may dominate and economy may be achieved by using inclined reinforcement. The increase in pure shear strength to be gained in this manner is given in Figure 5A-1.4.

It should be noted (see Fig. 5A-1.4) that if inclined steel is used, it is normally required in reasonably large amounts since the shearing deformation required to develop the steel strength is sufficient to reduce the shear strength of the concrete below its maximum value. For example:

Assume $\varphi'_v = 0.5$

Then $\varphi'_v(f_{dy}/f'_c) = 0.5\ (\frac{52{,}000}{3000}) = 8.65$

From Fig. 5A-1.4, $\lambda_s = 0.95$

which means that the pure shear strength of the concrete is such that if exceeded, the steel would be incapable of resisting the force that would be imposed upon it.

Diagonal Tension

Since there is to be no web reinforcement use $\mu = 1.3$

Fig. 5A-1.5, $\lambda_v = 1.0$; Fig. 5A-1.8, $\lambda_r = 0.70$

Fig. 5A-1.6, $\lambda_f \varphi_c f'_c = 3150$

Read, $d/L = 0.17$

Thus diagonal tension controls.

Required $d = 0.17 \times 12 \times 12 =$ 24.5 in.

Note that a smaller flexural steel percentage could have been used.

To illustrate the determination of diagonal tension resistance with web reinforcement assume that in the preceding example a web reinforcement percentage (φ_v) of 0.5 had been provided.

Diagonal Tension

Use $\mu = 3.0$; Fig. 5A-1.8, $\lambda_f = 0.70$

Fig. 5A-1.5, $\varphi_v f_{dy} = 26{,}000$ psi

Read, $\lambda_v = 1.23$

Fig. 5A-1.7, $\lambda_f \varphi_c f'_c = 3150$

Read $(\lambda_v d/L) = 0.14$

$$\text{Reqd. } \frac{d}{L} = \frac{0.14}{1.23} = 0.11$$

Thus the pure shear and diagonal tension requirements are now equal and the required $d = 0.11 \times 12 \times 12 = 16$ in.

The validity of the pure shear and diagonal tension computations should be checked by Figs. 5A-1.9 or 5A-1.10, whichever is applicable. For a d/L of 0.11 as chosen above, Fig. 5A-1.10 yields $p_m = 18$ psi. Since the actual flexural strength of the slab is greater than this, the computations are valid.

This example ignores the dead weight of the slab which is permissible in that case. However, if the structure had been buried it would have been necessary to include the dead weight of the earth. This may be done by converting the soil weight into an equivalent blast pressure. If the soil weighs 120 lb. per cu. ft., the pressure on the slab is 0.83 psi per ft. of depth. However, the charts include an increase in the applied pressure due to rapid loading of 1.20 for $\mu = 3.0$ and 1.625 for $\mu = 1.3$. Thus the equivalent blast pressure (psi) would be $0.83/1.2 = 0.69$ times the depth of earth in feet for $\mu = 3.0$ and $0.83/1.625 = 0.51$ times the depth of earth in feet for $\mu = 1.3$. These equivalent pressures would simply be added to the design blast pressure before entering the charts.

It is intended that the charts for one-way slabs also be used for walls. In the case of aboveground walls the value of p_m in the charts becomes the reflected pressure (see APPENDIX 5D). Since the reflected pressure is of short duration and the charts assume an infinite load duration there is obviously some error in this application. However, the errors are on the conservative side and would normally not exceed 25 percent.

5A.3.2 Two-way Slabs. Figure 5A-2.1 gives a factor, Ω, by which the flexural resistance of a one-way slab is multiplied to obtain the resistance of a two-way slab. This factor depends upon the ratio of the two side dimensions of the slab (α) and the positive and negative steel percentages in both directions.

The shear resistance of a two-way slab may be taken as $(2/3)(1 + \alpha)$ times that for a one-way slab spanning the short direction when α is greater than 1/2 and the same as a one-way slab when α is less than 1/2.

As an example determine the blast resistance of a simply-supported, 15 x 20 foot two-way slab having the following known properties (use $\mu = 1.3$).

Given:

$f'_c = 3000$ psi, $f'_{dc} = 3750$ psi, $f_{dy} = 52{,}000$ psi

$\varphi_c = 1.5\%$, $\varphi_e = 0$, short direction

$\varphi_c = 1.0\%$, $\varphi_e = 0$, long direction

$d = 10$ in.

Flexure:

Fig. 5A-2.1, $\alpha = \frac{15}{20} = 0.75$

$$\frac{\varphi_{LC} + \varphi_{LE}}{\varphi_{SC} + \varphi_{SE}} = \frac{1.0 + 0}{1.5 + 0} = 0.67$$

Read, $\Omega = 2.0$

Fig. 5A-1.1

$$(\varphi_c + \varphi_e)f_{dy} = (1.5 + 0)\ 52{,}000 \text{ (short direction)}$$
$$= 78{,}000 \text{ psi}$$

$$\frac{d}{L} = \frac{10}{15 \times 12} = 0.055$$

Read, $p'_m = 11$ psi (for one-way slab)

$\therefore\ p_m = 11 \times 2.0 = 22$ psi

Pure Shear:

Fig. 5A-1.3, $\frac{d}{L} = 0.055$, $f'_c = 3000$ psi

Read, $p_m = 50$ psi (for one-way slab)

$$\frac{2}{3}(1 + \alpha) = \frac{2}{3}(1 + 0.75) = 1.17$$

$\therefore\ p_m = 50 \times 1.17 = \underline{58 \text{ psi}}$

Diagonal Tension:

No web reinforcement, Fig. 5A-1.5, $\lambda_v = 1.0$;

Fig. 5A-1.8, $\lambda_f = 0.11$

Fig. 5A-1.6, $\lambda_f \varphi_c f'_c = 495$; $\lambda_v \frac{d}{L} = 0.055$

Read, $p_m = 4.4$ psi (for one-way slab)

$\therefore \quad p_m = 4.4 \times 1.17 = \underline{5.2 \text{ psi}}$

Thus diagonal tension controls and the blast resistance is only 5.2 psi.

5A.3.3 R/C Beams Supporting Slabs. The resistance of R/C beams is given by Figs. 5A-3.1 through 5A-3.9. These charts are similar to those for slabs. Note that p_m is the peak pressure (psi) on the slab being supported. Figures 5A-3.5 and 5A-3.9 provide factors by which the resistance of beams on the long side of non-square two-way slabs may be determined. The beams on the short side of such slabs are considered to be the same as beams of the same span under square slabs.

5A.3.4 Steel Beams. The flexural resistance of steel beams is given in Figs. 5A-4.1 through 5A-4.4 in terms of the section modulus of the cross section and the span. The shearing resistance is given in Figs. 5A-4.5 and 5A-4.6 in terms of the depth-span ratio and the web thickness. For beams supporting non-square two-way slabs Figs. 5A-3.5 and 5A-3.9 are used as for concrete beams.

5A.3.5 Flat Slabs. The flexural resistance of flat slabs as given by Figs. 5A-5.1 and 5A-5.2 depends upon the factors X and X' given by Fig. 5A-5.3 and K given by Fig. 5A-5.4. In these charts the parameter Φ is an effective total of the steel percentages in the two directions and is determined by the equation given on the charts.

The shear resistance of flat slabs is given by Fig. 5A-5.5 using the parameters K_1 and K_2 given by Fig. 5A-5.6 and the flexural resistance given by the preceding charts. Thus it is necessary to determine the flexural resistance before computing the shear resistance. It is also necessary to consider two possibly critical sections, one around the column capital and the other around the drop panel.

As an example the blast resistance of a square flat slab system is computed below:

Given:

Span = $L_1 = L_2 = 20$ ft.

Width of drop panel = $p_1 = p_2 = 7$ ft.

Width of column capital = $c_1 = c_2 = 4$ ft.

Effective depth of slab = d = 16 in.

Effective depth of drop panel = d_p = 32 ins.

Steel percentages (both directions)

Bottom of slab at mid-span, $\varphi_{b1} = \varphi_{b2} = 1.0$

Top of slab at midspan, $\varphi_{t1} = \varphi_{t2} = 1.25$

Top of drop panel
(computed on the basis of d_p) $\varphi'_{t1} = \varphi'_{t2} = 1.5$

f_{dy} = 52,000 psi

f'_c = 4000 psi, f'_{dc} = 5000 psi

No web reinforcement

Use $\mu = 3.0$ in flexure and $\mu = 1.3$ in shear

<u>Flexural Resistance</u>:

Fig. 5A-5.3, $p/L = 0.35$, $d_p/d = 2.0$

Read, $X = 0.65$, $X' = 1.40$

Fig. 5A-5.4, $c/L = 0.2$, $L_1/L_2 = 1.0$

Read, $K = 1.34$

Fig. 5A-5.2

$$\varphi = \varphi_{b1} + \varphi_{b2} + X(\varphi_{t1} + \varphi_{t2}) + X'(\varphi'_{t1} + \varphi'_{t2})$$

$$= 1.0 + 1.0 + 0.65(1.25 + 1.25) + 1.40(1.5 + 1.5) = 7.83$$

$f_{dy}\,\varphi = 407{,}000$

$$\frac{d}{L} = \frac{16}{20 \times 12} = 0.067$$

Read, $p_{mf}K$ = 110 psi

$$p_{mf} = \frac{110}{1.34} = \underline{82 \text{ psi}}$$

<u>Shear resistance</u>:

Fig. 5A-5.6, $c/L = 0.2$, $L_2/L_1 = 1$, $p/L = 0.35$

Read, $K_1 = 2.4$

$K_2 = 1.25$

Fig. 5A-5.5, $f'_c\, p_{mf} = 3000 \times 69 = 207{,}000$

Shear around drop panel, $\frac{d}{LK_2} = \frac{16}{20 \times 12 \times 1.25} = 0.053$

Read, p_{mv} = <u>35 psi</u>

Shear around column capital, $\frac{d_b}{LK_1} = \frac{32}{20 \times 12 \times 2.4} = 0.056$

Read, p_{mv} = 35 psi

Thus the blast resistance is 35 psi.

5A.3.6 Aboveground Rectangular Buildings. Figures 5A-6.1 and 5A-6.2 provide the resistance of rigid building frames in terms of the parameter $\Sigma M/bh^2$ where ΣM is the sum of both end-moment capacities of all columns (or connections) in the frame. The abscissa of the plot is the side-on pressure p_{so} but the reflected pressure on the front face and the drag force on the building have been taken into account. The parameter $(Y + 3h)/T_n$ accounts for the effect of duration of the reflected pressure. The natural period T_n may be computed (see APPENDIX 5B) or may be taken as 0.1 sec. which is probably a lower (and hence conservative) limit for such frames. It should be noted that rigid frames will not carry the blast load on a building if shear walls which are normally stiffer are also present.

As an example consider a building with four 20-foot bays and 15 feet high with frames spaced at 25 feet. All five columns are to be identical and rigidly connected top and bottom. Allowing a ductility ratio of 3.0 it is desired to determine the required column strength for a peak side-on pressure of 20 psi. Lacking additional information T_n may be taken as 0.1 sec. and $\frac{Y + 3h}{T_n} = \frac{80 + (3 \times 15)}{0.1} = 1250$ ft./sec. Reading Fig. 5A-6.2 it is found that $\Sigma M/bh^2$ is 24 psi. Thus ΣM must be $24 \times (25 \times 12) \times (15 \times 12)^2/1000 = 233{,}300$ kip-in. Since there are 10 rigidly connected column ends the required bending strength of each column is 23,330 kip-ins. The columns may then be designed according to APPENDIX 5B or Figs. 5A-10.1 through 5A-10.3. The vertical column load should be superimposed on this bending moment but in most cases neglecting this load would not cause serious error. In fact for concrete columns it is usually conservative to ignore the vertical load.

For buildings in which the lateral resistance is provided by shear walls the required horizontal cross-sectional area of the walls is given by Figs. 5A-6.3 and 5A-6.4. These charts are based upon the assumption that shear walls are relatively stiff and hence the natural period of the building is short compared to the duration of the load. It is also assumed that the ultimate resistance of the wall is equal to the cracking strength. This requires approximately one percent steel in each direction.

5A.3.7 R/C Arches. For purposes of analysis arches are divided into three categories: fully-buried, partially-buried and aboveground. For an arch to be considered fully-buried the average depth of cover (H_{av}) must be at least 0.25 times the span (B) and, in addition, the depth of cover at the crown (H_c) must be at least 0.125B. If cover is provided by an embankment above the general ground level, the surface should be horizontal above the arch and the slope of the fill outside the foundation must not be steeper than 1 on 4. A partially-buried arch is defined as one for which H_c

is between zero and 0.062B. If any portion of the arch projects aboveground or if it is covered by an embankment not meeting the requirements given above, it should be considered completely aboveground. More refined methods for handling this case are given in Ref. 5D-6.

A linear interpolation of required thickness may be used between the limit of the partially-buried case ($H_c = 0.062B$) and the fully-buried case (see example). The required thickness of fully-buried circular arches may be determined by Figs. 5A-7.1 through 5A-7.3. The required ratio of total thickness to radius is given as the sum of that required for dead load and that for blast load. The weight of soil was taken as 120 lb. per cu. ft. Note that the abscissa, p_m, is the peak vertical pressure at the average arch depth which is the surface pressure attenuated with depth.

The required thickness of partially-buried arches is given by Figs. 5A-7.4 or 5A-7.5. The effect of the transit of the shock front on the flexural stresses has been included. The effect of dead load may be approximately included by simply increasing p_{so} by the average weight of arch and cover.

The required thickness of aboveground arches is given by Figs. 5A-7.6 or 5A-7.7. Flexural stresses due to transit of the shock front and drag pressures have been included. Interpolation for values of the rise-span ratio (h/B) is necessary. Approximate correction factors are given to account for variations in material strength and steel percentage.

In addition to the strength requirement given by the charts, the buckling stability of the arch must be considered. This criterion is satisfied if p_{so} is less than $(1 - \frac{\beta^2}{\pi^2}) \cdot \frac{\pi^2 ED^3}{18r^3\beta^2}$. This expression should be used for aboveground structures but buckling need not be considered in the fully-buried case. For partially buried arches this value may be increased by 50 percent. If the arch thickness given by the charts does not satisfy the buckling criterion this thickness must be increased.

If H_{av} is greater than one-half the span B, the arching effect of the soil to carry the blast load may be considered. In such cases it is recommended that the value of p_m be the peak surface blast pressure reduced by $\frac{H_{av} - B}{B} \cdot \sigma_s$ where σ_s is the shearing strength of the soil due only to friction. However, p_m should never be taken as less than 0.25 times the peak surface pressure. This procedure may also be used for domes and steel arches.

As an example consider blast resistance for $\mu = 1.3$ of a buried reinforced concrete barrel arch having the following characteristics:

Given:

Arch radius = r = 40 ft.

Half internal angle = β = 60° = $\frac{\pi}{3}$

Total arch thickness = D = 2 ft.

Depth of arch to steel = d = 21 in.

Depth of earth cover at crown = H_c = 6 ft.

Steel percentage, one face = Φ = 1.0

Total steel percentage = Φ_T = 2.0

f'_{dc} = 5000 psi, f_{dy} = 52,000 psi

Fully-Buried Criterion:

Span = B = 2r sin β = 69.2 ft.

Rise = r(1 - cos β) = 20 ft.

H_{av} = average depth of cover

$$= \left[B(\text{rise} + H_c) + \frac{1}{2}B(r\cos\beta) - \beta r^2\right]\frac{1}{B}$$

$$= (20 + 6) + \frac{1}{2}(20) - \frac{\pi}{3}(40)^2\frac{1}{69.2} = 11.8 \text{ ft.}$$

or, H_{av} = 0.170B $\quad$ H_c = 0.087B

Since H_{av} is less than 0.25B and H_c is less than 0.125B but greater than 0.062B, the arch is not fully buried and the resistance must be determined by interpolation between the partially-buried and fully-buried cases.

Fully-Buried Case:

Fig. 5A-7.1, $(0.85\, f'_{dc} + 0.009\, f_{dy}\, \Phi_T)$ = 5190

H_{av} + D = 13.8

Read, $\left(\frac{D}{r}\right)_{H_{av}+D}$ = 0.0023

Fig. 5A-7.2

$$\left(\frac{D}{r}\right)_{p_m} = \frac{D}{r} - \left(\frac{D}{r}\right)_{H_{av}+D} = \frac{2}{40} - 0.0023 = 0.0477$$

Read, p_m = <u>148 psi</u> = p_{so} (at this shallow depth $p_m = p_{so}$)

Partially-Buried Case:

Fig. 5A-7.4, $\Phi f_{dy} = 52{,}000$

Read, $\dfrac{d}{rp_{so}} = 0.00034$

$$p_{so} = \frac{21}{40 \times 12 \times 0.00034} = \underline{128 \text{ psi}}$$

Interpolation:

$$p_{so} = 128 + (148 - 128)\,\frac{0.087-0.062}{0.166-0.062} = 133 \text{ psi}$$

Note that for $\beta = \pi/3$, H_c must equal 0.166B in order to make H_{av} equal to 0.25B as required for full burial.

Buckling Criterion:

$$\text{Critical } p_{so} = \frac{3}{2}\left(1 - \frac{\beta^2}{\pi^2}\right) \frac{\pi^2 ED^3}{18r^3\beta^2} \text{ (partially-buried case)}$$

$$= \frac{3}{2}\left(1 - \frac{1}{9}\right) \frac{\pi^2 \times 3 \times 10^6 \times 2^3}{18(40)^3(\pi/3)^2} = \underline{250 \text{ psi}}$$

Buckling is not critical since 250 > 128. If this value had been less than 128 psi the interpolation above should be repeated substituting this value for 128 psi.

Thus the blast resistance (which in this case is side-on overpressure) for this arch is 133 psi.

As a further example consider an arch having the same properties as that above except that it is completely aboveground.

Using Fig. 5A-7.6:

$$\frac{h}{B} = \frac{r(1 - \cos\beta)}{2r \sin\beta} = 0.29$$

$$\frac{D}{B} = \frac{2}{69.2} = 0.029$$

Read: $P_{so}/CF = 30$, interpolating

$CF = 1.02$, interpolating

$\therefore \quad P_{so} = (30)(1.02) = 31$ psi

= Arch Resistance

5A.3.8 R/C Domes. The required thickness of fully-buried domes may be taken as one-half that of an arch with the same loading and radius (Figs. 5A-7.1, 5A-7.2 and 5A-7.3). The required thickness of partially-buried domes is given by Figs. 5A-8.1 and 5A-8.2 and that for aboveground domes by Figs. 5A-8.3 and 5A-8.4. The cover requirements for full and partial burial and the interpolation between these cases is the same as given in SECTION 5A.3.7 for arches. The buckling requirement for domes is that p_{so} must not exceed $0.8ED^2/r^2$ for aboveground domes and three-halves of this value for partially-buried domes. Buckling need not be considered in the fully-buried case.

5A.3.9 Steel Arches. The required cross-sectional area of fully-buried steel arches is provided by Figs. 5A-9.1 and 5A-9.2 in the same manner as the required thickness of R/C arches. The required area for partially-buried arches may be determined by Figs. 5A-9.3 and 5A-9.4 and for aboveground arches by Figs. 5A-9.5 and 5A-9.6. In the latter cases the steel yield strength has been taken as 42,000 psi. The required area may be assumed to vary inversely with yield strength.

The definitions of the aboveground, partially-buried and fully-buried cases and the method of interpolation are the same as given above for R/C arches. However, buckling requirements have been included in the charts for steel arches and no separate calculations are necessary.

As an example of aboveground steel arch design, consider a structure consisting of 36 WF 300 arch ribs (or equivalent built-up sections) with a non-composite cover, a radius of 40 feet and a rise-span ratio of 0.3. It is desired to find the required spacing of ribs for $p_{so} = 50$ psi and $\mu = 1.3$. Making use of the properties of the Tee-section cut from 36 WF 300 section as given in steel handbooks, the dimension d_z is the distance from the bottom to the center of gravity of the Tee or 14.23 in. Thus r/d_z is $40 \times 12/14.23 = 34$ and, for Fig. 5A-9.5, $A/r\,p_{so}$ is found to be 0.000162 (interpolating between 30 and 100 psi). The required area per inch of width is therefore $0.000162 \times 50 \times 40 \times 12 = 3.9$ ins. Since the area of a 36 WF 300 section is 88 sq. in., the required spacing is 88/3.9 or 23 ins.

5A.3.10 Columns. The ultimate dynamic strengths of axially-loaded reinforced concrete and steel columns may be determined by Figs. 5A-10.1 and 5A-10.3. The latter gives the strength of steel columns in terms of the flange width and weight per foot. It is applicable to all standard WF sections without appreciable error. If the column supports a roof subjected to blast loading the column load should be taken as twice the peak blast pressure times the tributary area or the maximum resistance of the supported elements, whichever is smaller.

Fig. 5A-10.2 may be used to determine the strength of reinforced concrete beam-columns or members subjected to combined bending and direct compression. In the chart the ordinate is the ratio of axial load to ultimate compressive strength as given by Fig. 5A-10.1. The abscissa is the ratio of moment to ultimate bending strength as given in APPENDIX 5B. The strength of steel beam-columns may be determined using the criterion of APPENDIX 5B.

5A.3.11 Footings. The strength of square column footings may be found by use of Figs. 5A-11.1 and 5A-11.2. These may be used to determine the required thickness or steel percentage for a given soil bearing pressure after the plan size has been determined from the soil characteristics. Such footings should be designed for the maximum dynamic column load. The allowable dynamic soil pressure to be used in determining the required width of footing is given in APPENDIX 5B.

The strength of wall footings (or footings under arches or domes) may be determined either by Fig. 5A-11.3 or Fig. 5A-11.4 depending upon which provides the smaller reading. As in the case of column footings, these charts give the bearing strength based upon the footing itself and not on the soil characteristics. The allowable soil strength is given in APPENDIX 5B. In the case of buried arches or domes, the arching effect of the soil above the structure usually may be taken into account when computing footing loads even though this is not done in designing the structure itself.

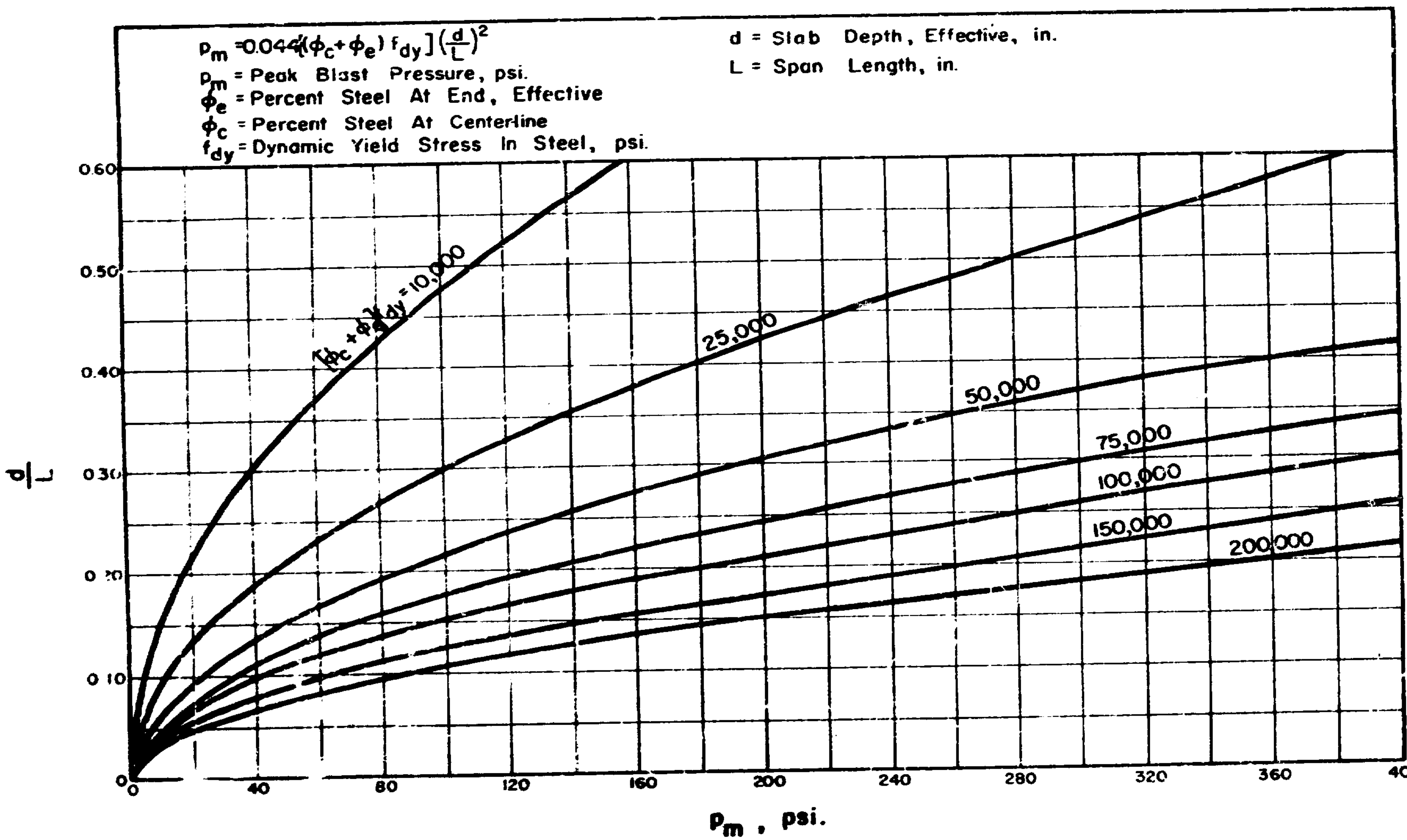


FIG. 5A-1.1 FLEXURAL RESISTANCE OF SIMPLY SUPPORTED AND CONTINUOUS ONE-WAY SLABS ($\mu = 1.3$)

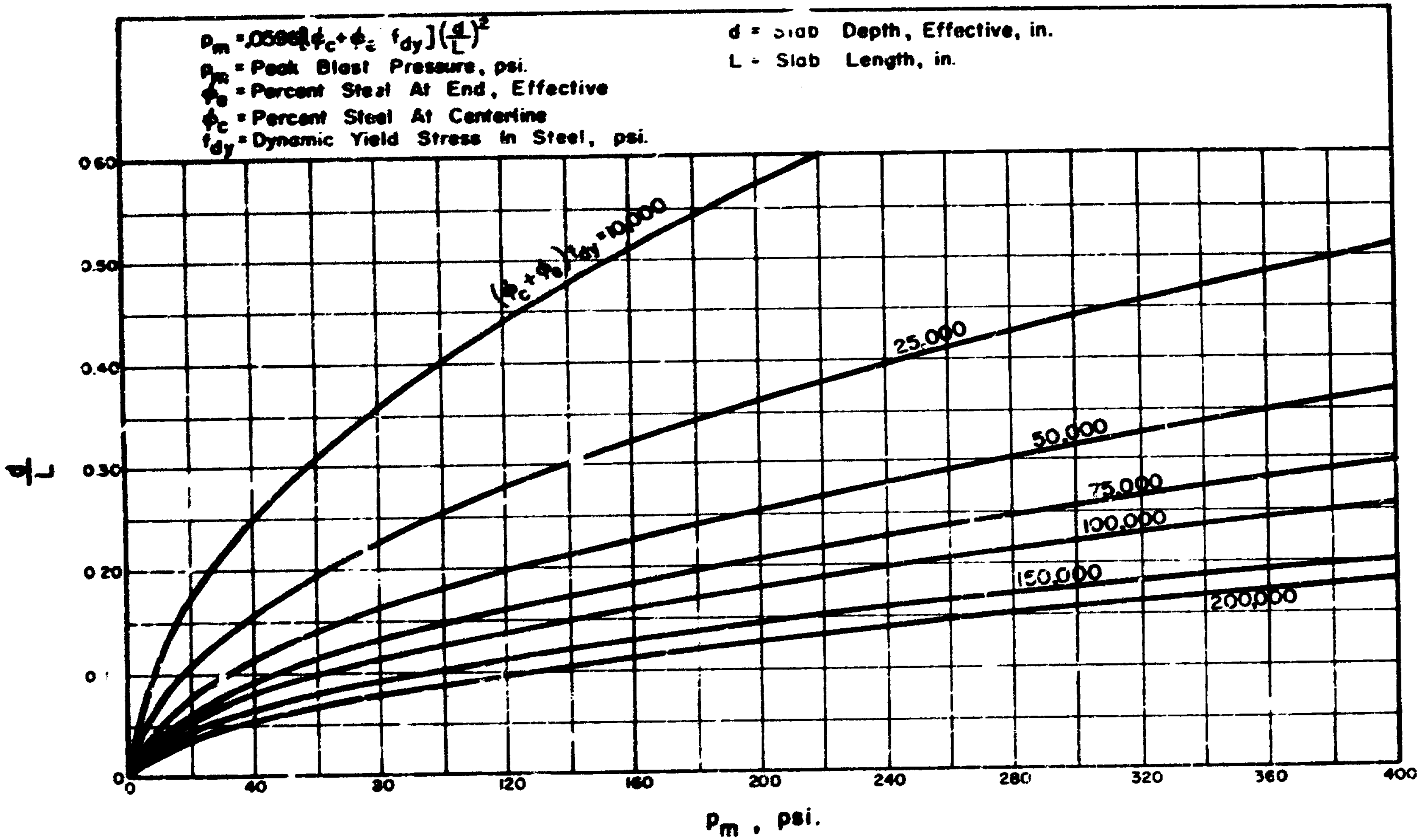


FIG. 5A-1.2 FLEXURAL RESISTANCE OF SIMPLY SUPPORTED AND CONTINUOUS ONE-WAY SLABS ($\mu = 3.0$)

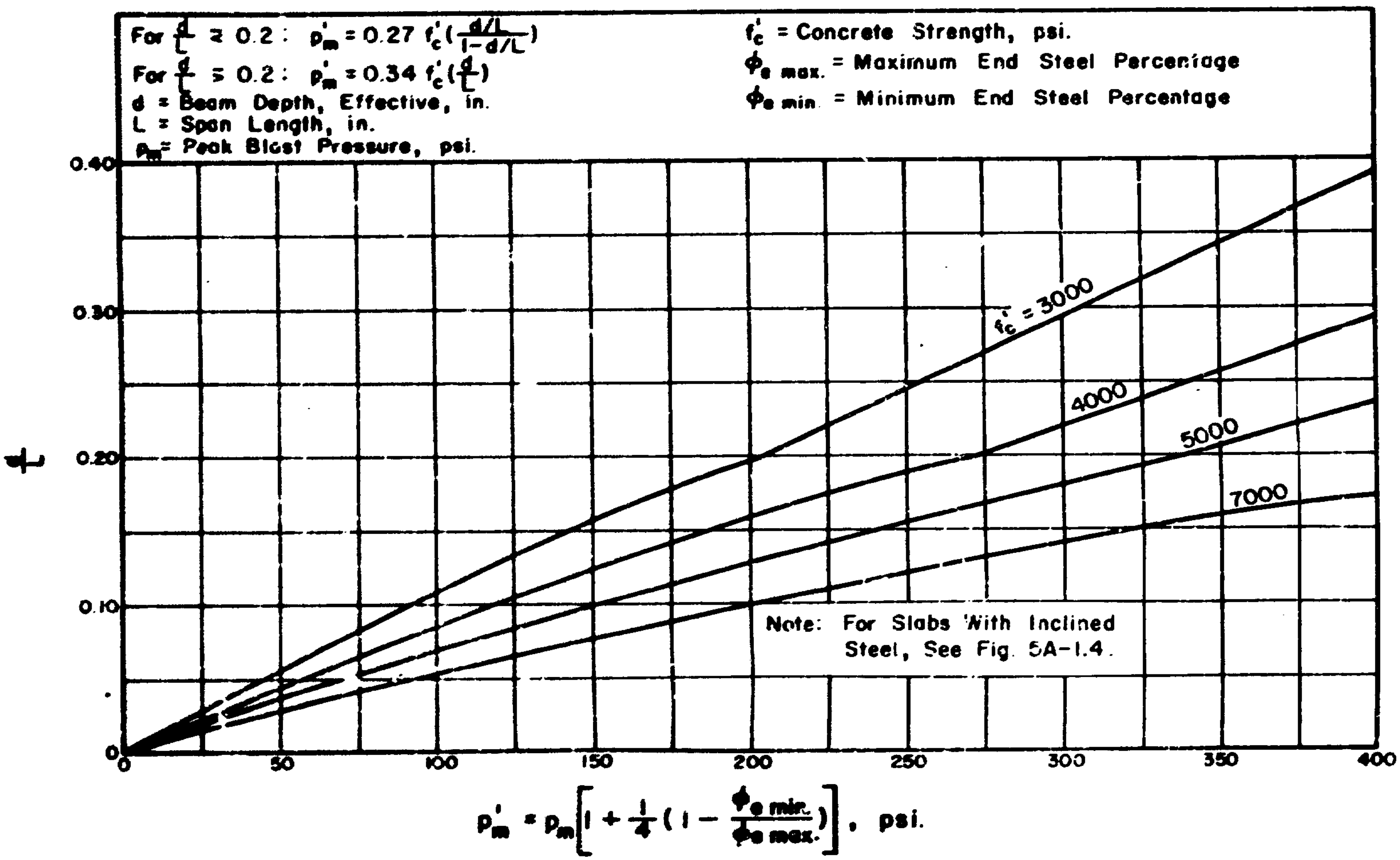
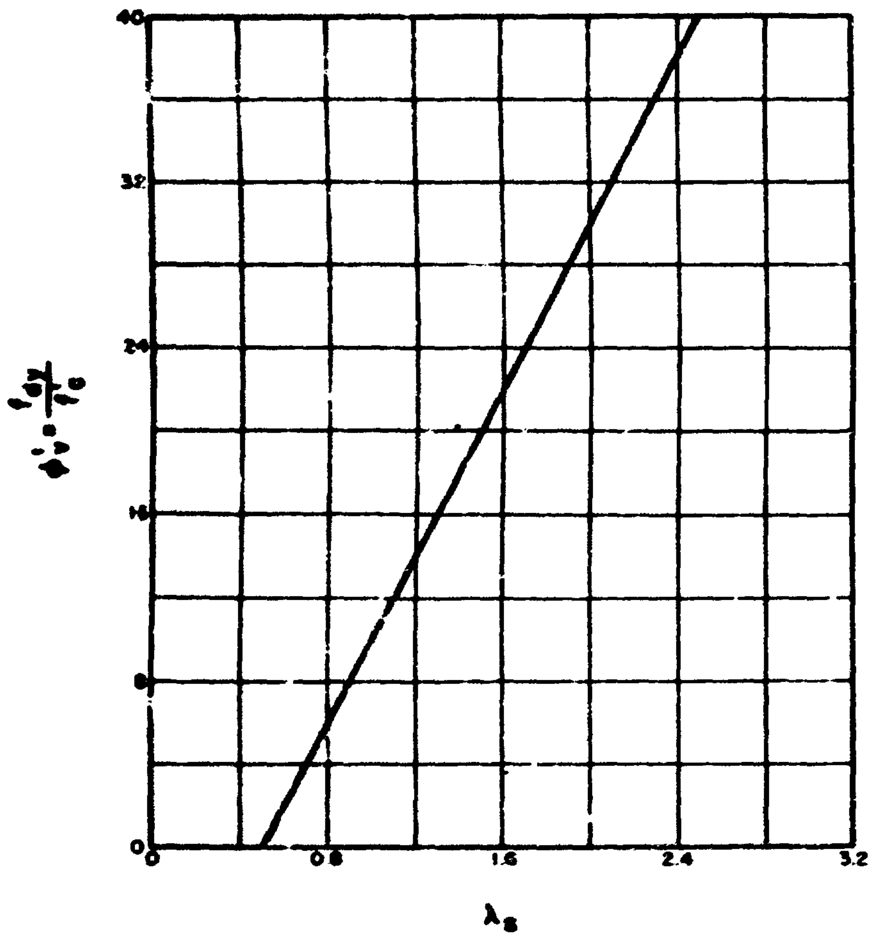


FIG. 5A-1.3 RESISTANCE IN PURE SHEAR OF ONE-WAY SLABS WITHOUT INCLINED WEB STEEL ($\mu = 1.3$)



$\lambda_s = \frac{1}{2}\left[1 + \frac{1}{10}\left(\phi'_v \frac{f_{dy}}{f'_c}\right)\right]$

λ_s = Pure Shear Factor; Ratio Of Pure Shear Resistance Of Slab or Beam With Inclined Steel To That Of Same Slab or Beam Without Inclined Steel.

f_{dy} = Dynamic Yield Stress In Steel, psi.

f'_c = Concrete Strength, psi.

ϕ'_v = Percent Of Inclined Steel Intersecting a 45° Surface Through Beam or Slab At End Of Beam or Slab. Express As Percentage Of Concrete Area On 45° Surface.

NOTE:

(1) To Obtain Resistance In Pure Shear, Consult First Fig. 5A-1.3 or 5A-3.6. Multiply Value Thus Obtained By λ_s If $\lambda_s > 1.0$.

(2) If λ_s Is less Than 1.0, The Inclined Steel Is Ineffective.

FIG. 5A-1.4 PURE SHEAR RESISTANCE — FACTOR FOR SLABS AND BEAMS WITH INCLINED STEEL AT ENDS

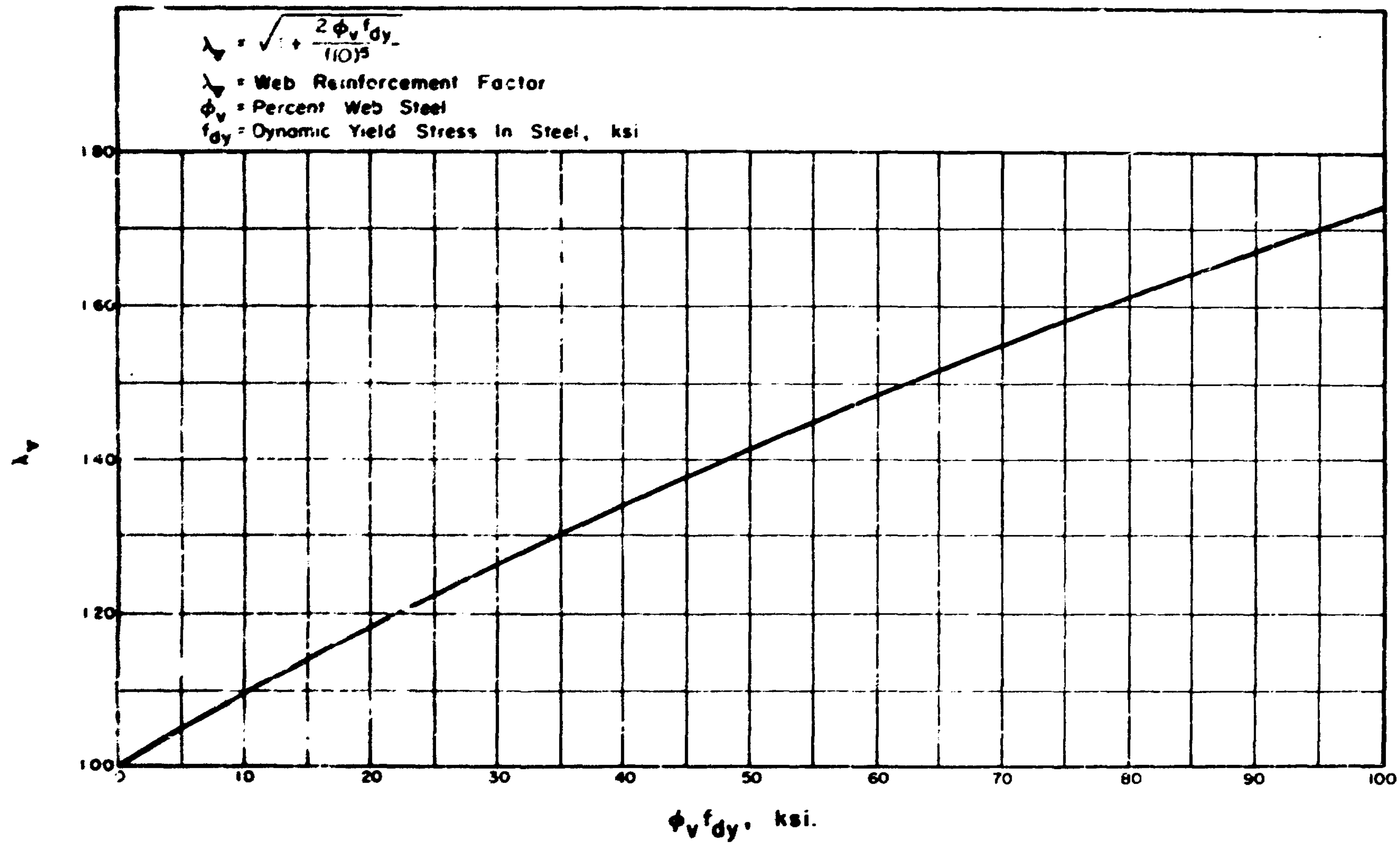


FIG. 5A-1.5 WEB REINFORCEMENT FACTOR

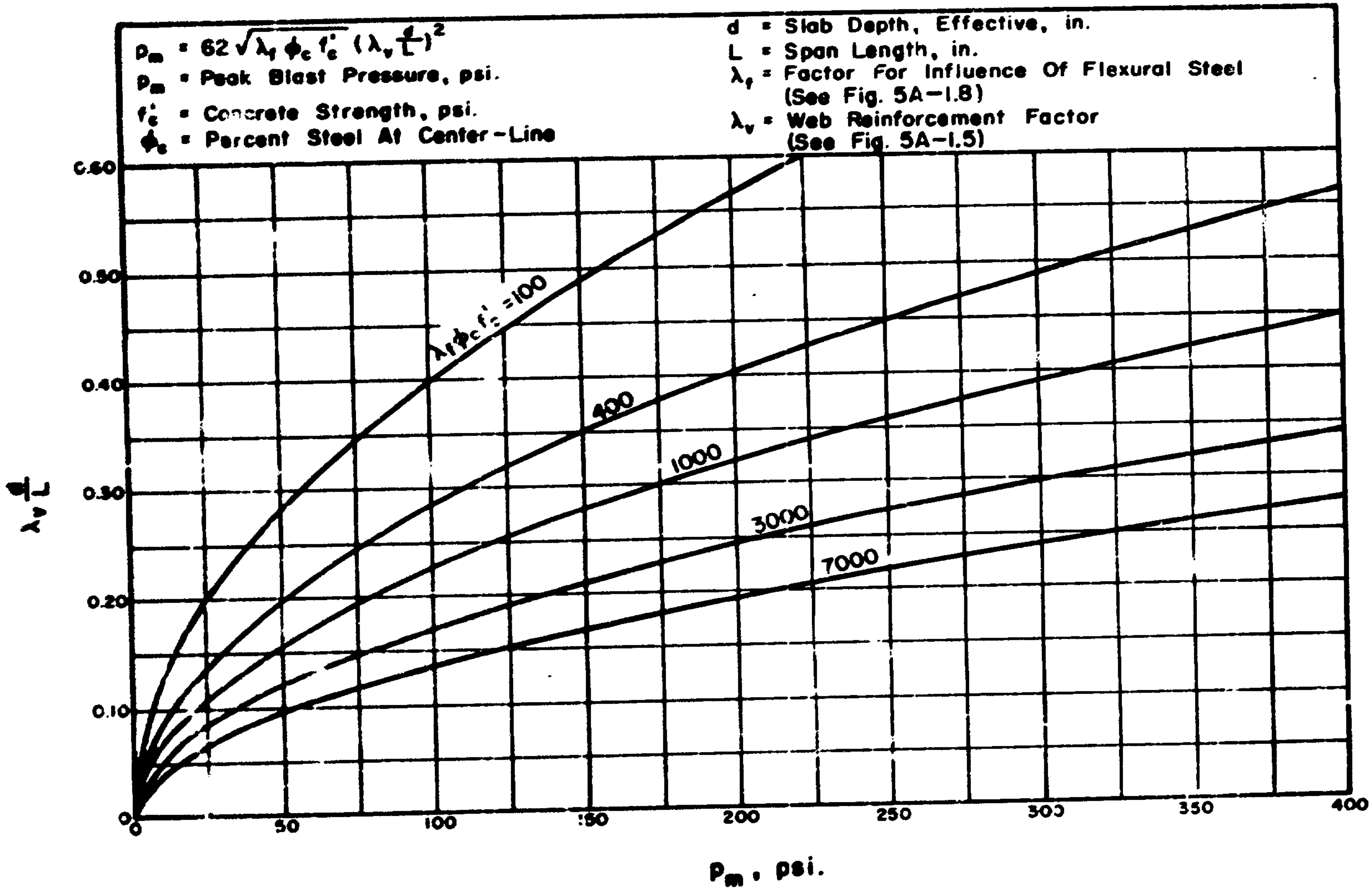


FIG. 5A-1.6 RESISTANCE IN DIAGONAL TENSION OF ONE-WAY SLABS
(μ = 1.3)

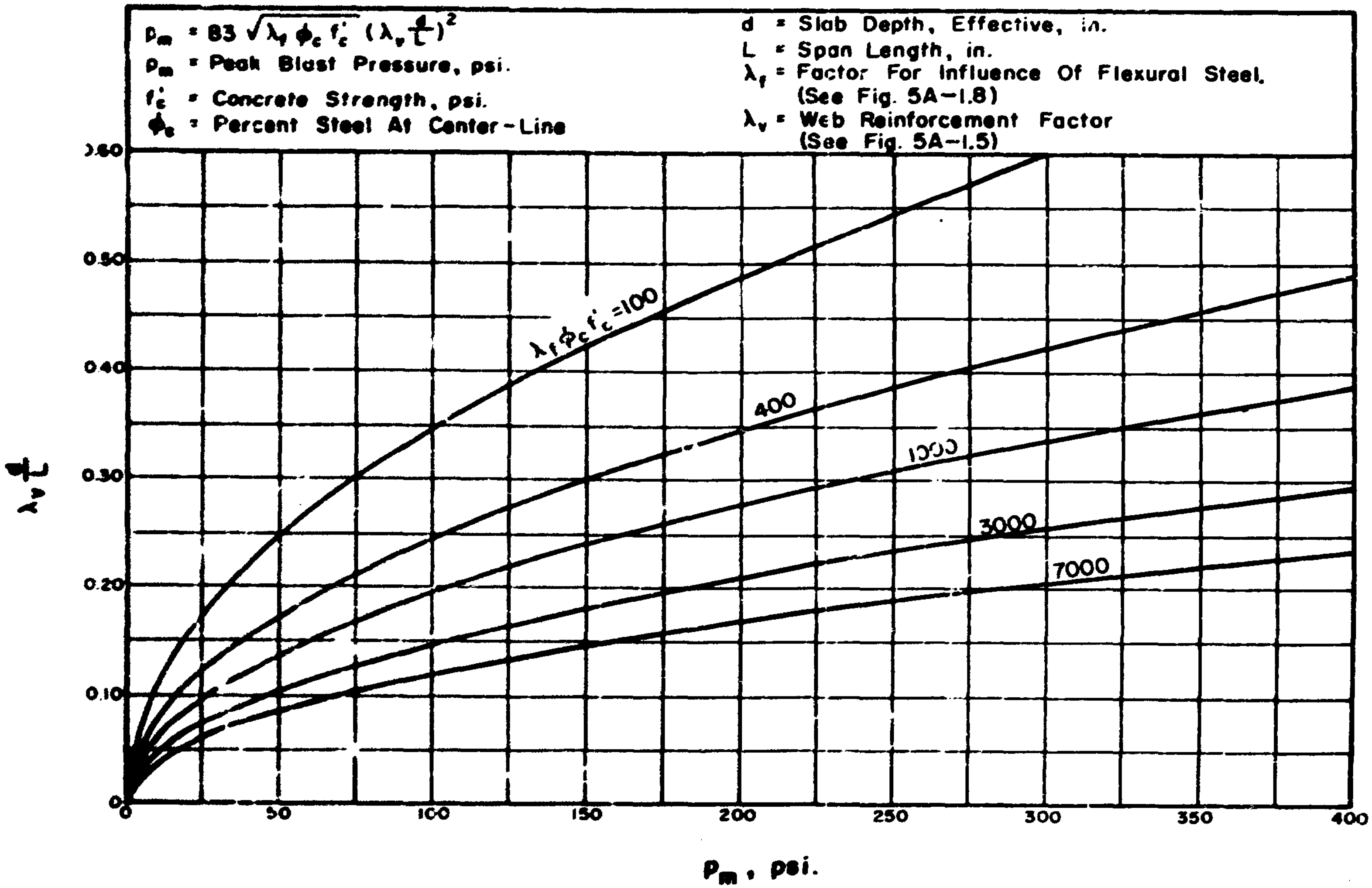
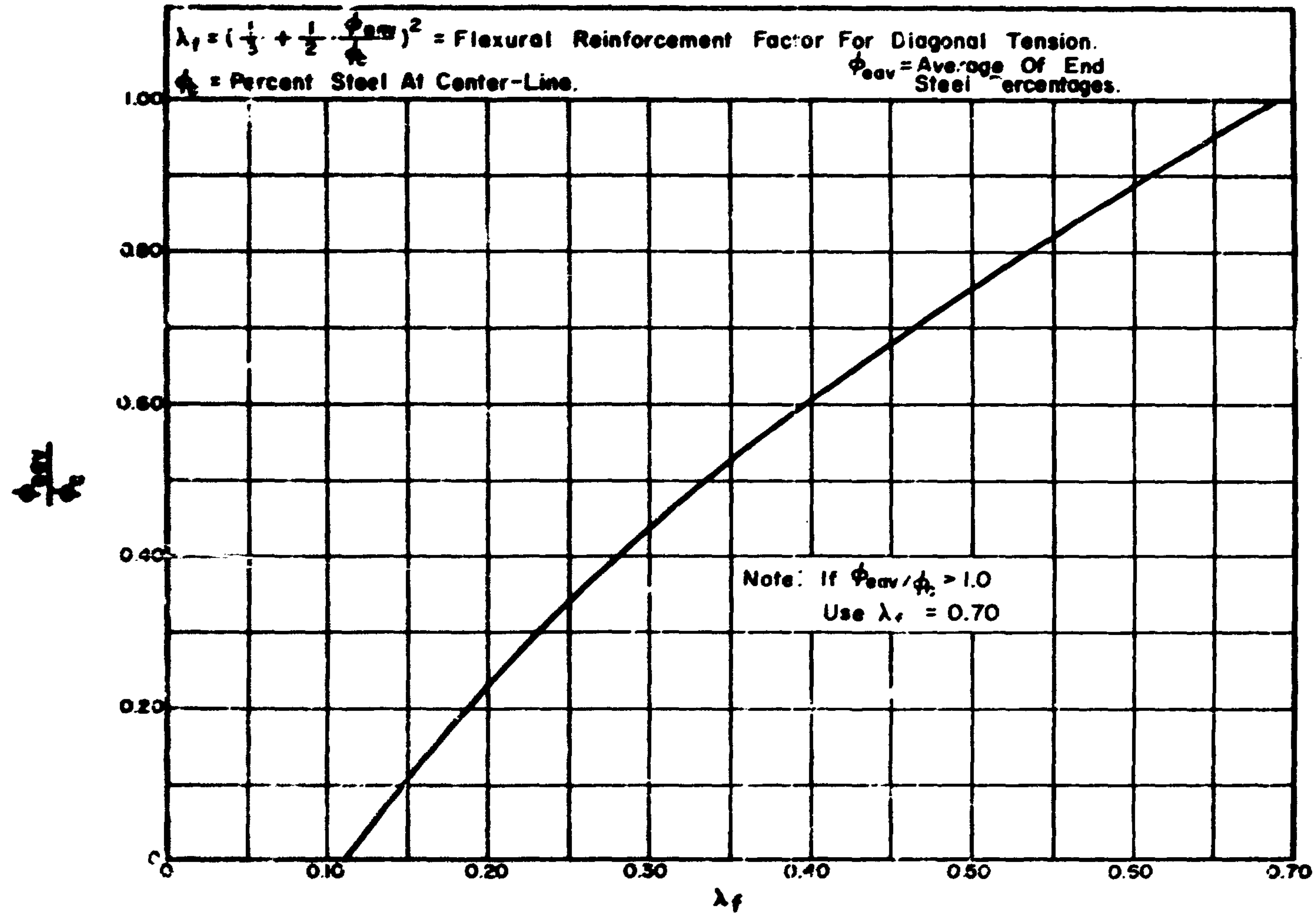


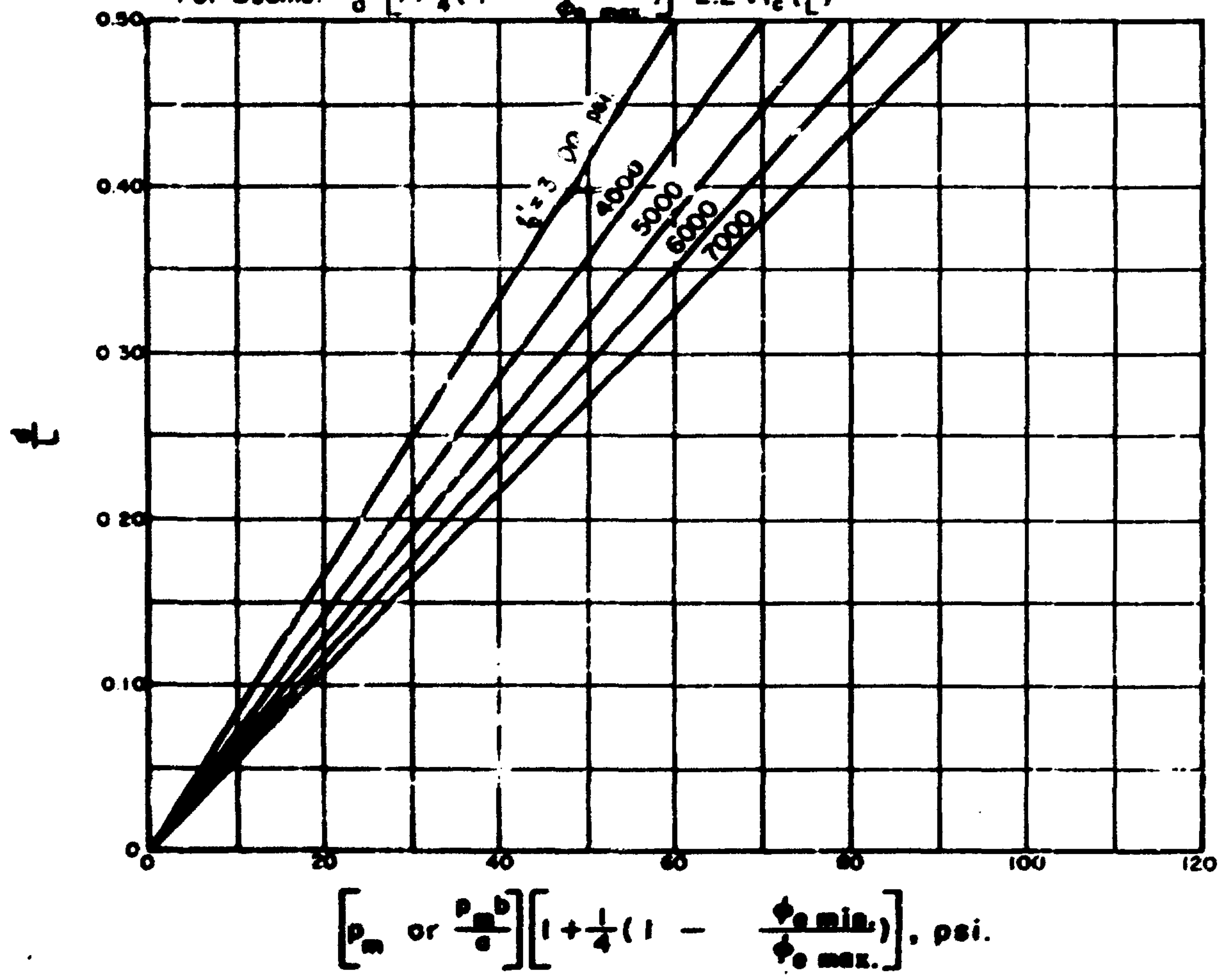
FIG. 5A-1.7 RESISTANCE IN DIAGONAL TENSION OF ONE-WAY SLABS
(μ = 3.0)



.'IG. 5A-1.8 RESISTANCE IN DIAGONAL TENSION FOR ONE-WAY SLABS AND BEAMS — FACTOR FOR INFLUENCE OF FLEXURAL STEEL.

For Slabs: $p_m\left[1+\frac{1}{4}\left(1-\frac{\phi_{e\,min.}}{\phi_{e\,max.}}\right)\right] = 2.2\sqrt{f'_c}\left(\frac{d}{L}\right)$

For Beams: $\frac{p_m b}{a}\left[1+\frac{1}{4}\left(1-\frac{\phi_{e\,min.}}{\phi_{e\,max.}}\right)\right] = 2.2\sqrt{f'_c}\left(\frac{d}{L}\right)$



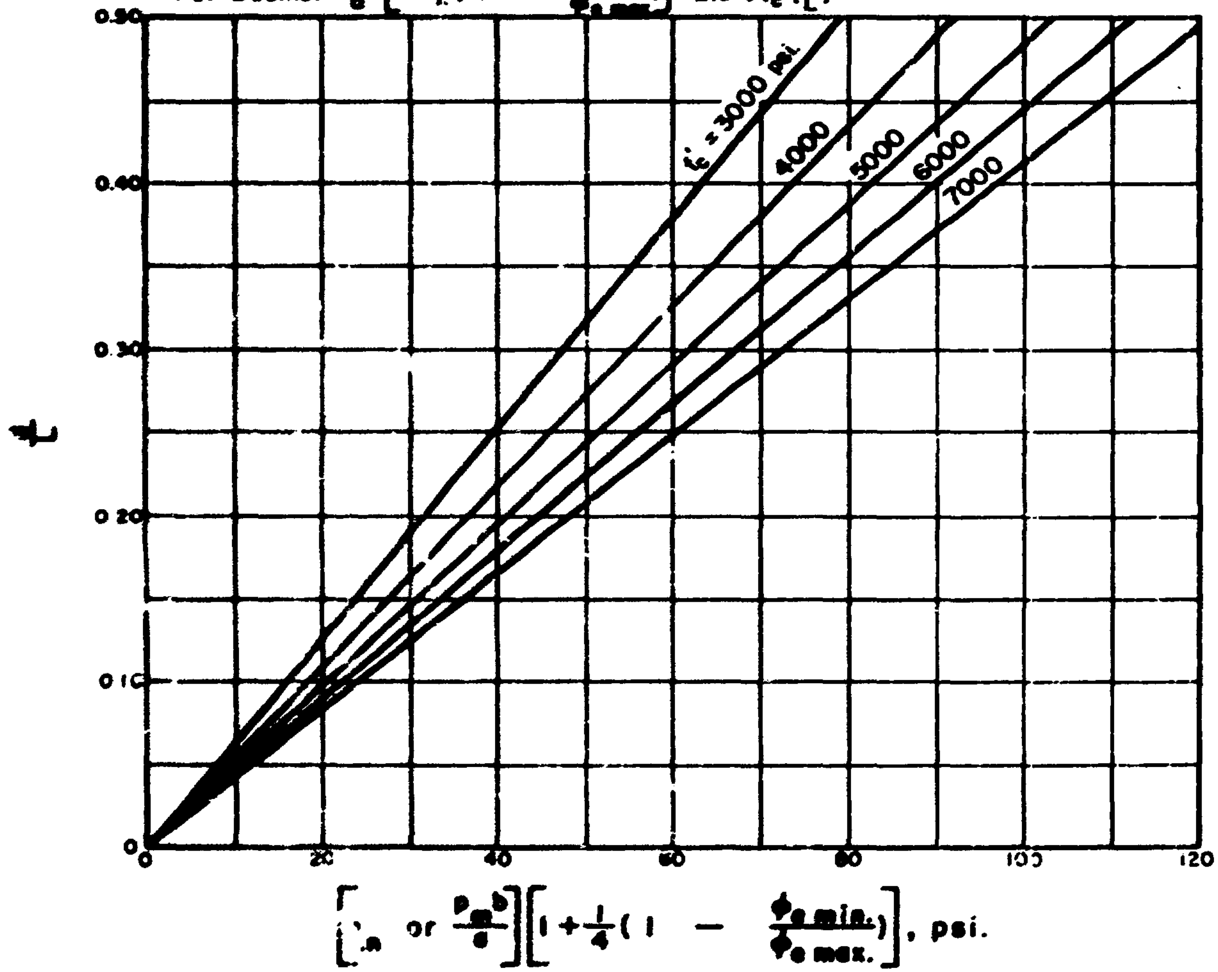
p_m = Peak Blast Pressure, psi.
b = Beam Spacing, in.
a = Beam Width, in.
$\phi_{e\,max.}$ = Larger Of Two End Steel Percentages.
$\phi_{e\,min.}$ = Smaller Of Two End Steel Percentages.
L = Span Length, in.
f'_c = Concrete Strength, psi.
d = Depth Of Section, Effective, in.

NOTE:
For a Given Slab Or Beam Having a Flexural Resistance Less Than That Given Here, Failure In Modes Other Than Flexural Is Impossible Regardless Of Data Given On Charts For Pure Shear And For Diagonal Tension.

FIG. 5A-1.9 LOWER LIMIT FOR NON-FLEXURAL FAILURE OF REINFORCED CONCRETE ONE-WAY SLABS AND BEAMS (μ=1.3)

For Slabs: $p_m\left[1+\frac{1}{4}\left(1-\frac{\phi_{e\,min.}}{\phi_{e\,max.}}\right)\right]=2.9\sqrt{f'_c}\left(\frac{d}{L}\right)$

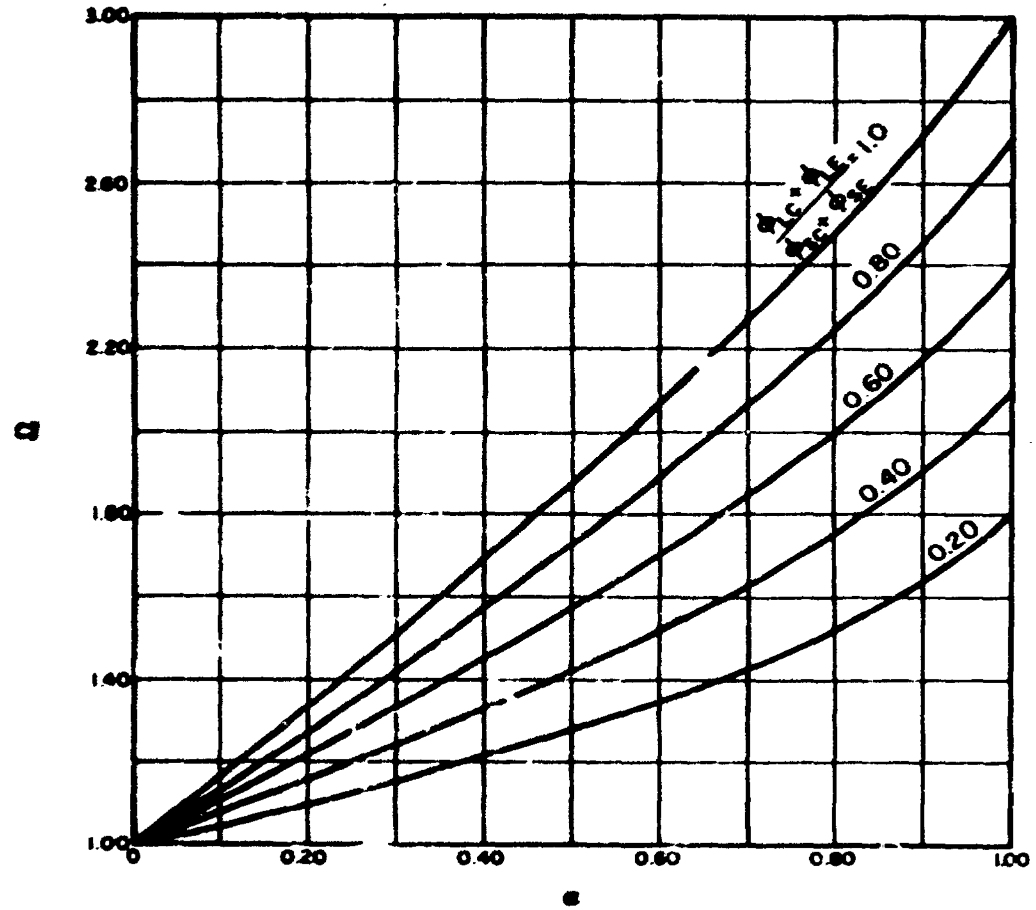
For Beams: $\frac{p_m b}{a}\left[1+\frac{1}{4}\left(1-\frac{\phi_{e\,min.}}{\phi_{e\,max.}}\right)\right]=2.9\sqrt{f'_c}\left(\frac{d}{L}\right)$



p_m = Peak Blast Pressure, psi.
b = Beam Spacing, in.
a = Beam Width, in.
$\phi_{e\,max.}$ = Larger Of Two End Steel Percentages.
$\phi_{e\,min.}$ = Smaller Of Two End Steel Percentages.
L = Span Length, in.
f'_c = Concrete Strength, psi.
d = Depth Of Section, Effective, in.

NOTE:
For a Given Slab Or Beam Having a Flexural Resistance Less Than That Given Here, Failure In Modes Other Than Flexural Is Impossible Regardless Of Data Given On Charts For Pure Shear And For Diagonal Tension.

FIG. 5A-1.10 LOWER LIMIT FOR NON-FLEXURAL FAILURE OF REINFORCED CONCRETE ONE-WAY SLABS AND BEAMS (μ= 3.0)



$$\Omega = 1 + \alpha \left[\frac{1}{6-4\alpha} + \frac{3}{2} \left(\frac{\phi_{LC} + \phi_{LE}}{\phi_{SC} + \phi_{SE}} \right) \right]$$

Ω = Two-Way Slab Factor

α = Ratio Of Short Side To Long Side

ϕ_{LC} = Percent Steel In Long Direction at Center-line

ϕ_{LE} = Percent Steel In Long Direction At End, Effect.

ϕ_{SC} = Percent Steel In Short Direction at Center-line

ϕ_{SE} = Percent Steel In Short Direction At End, Effective.

NOTE:

Flexural Strength Of Two-Way Slabs Can Be Determined As Follows:

(1) Treat As One-Way Slab Spanning Short Direction.

(2) Determine Corresponding Strength From Figures 5A-1.1 or 5A-1.2.

(3) Multiply Value From (2) above By Ω.

Restrict Relative Steel Percentages In Both Directions To Insure Development Of Yield Lines.

FIG. 5A-2.1 FLEXURAL RESISTANCE — TWO-WAY SLAB FACTOR

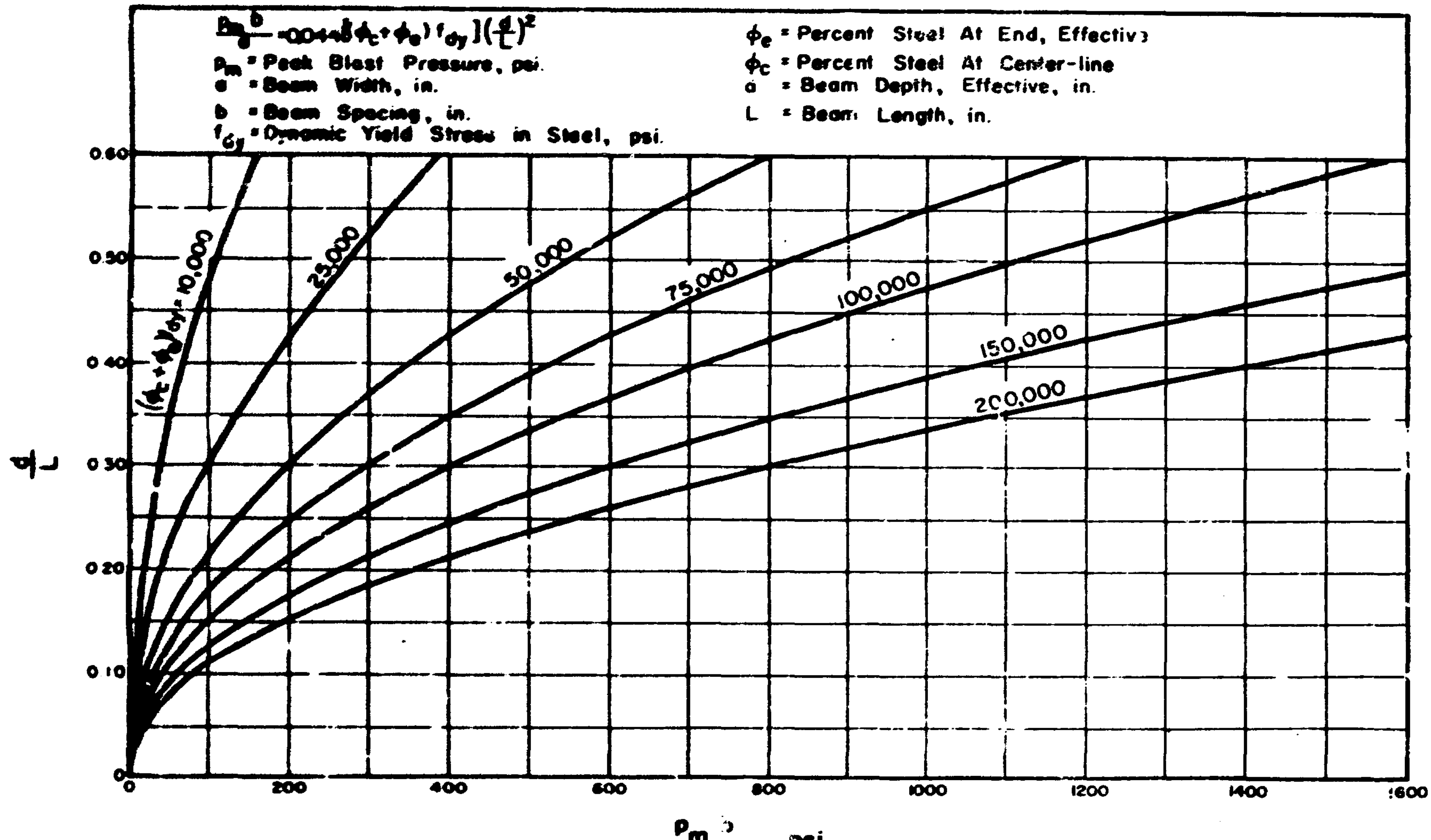


FIG. 5A-3.1 FLEXURAL RESISTANCE OF SIMPLY SUPPORTED OR CONTINUOUS REINFORCED CONCRETE BEAMS SUPPORTING ONE-WAY SLABS (μ = 1.3)

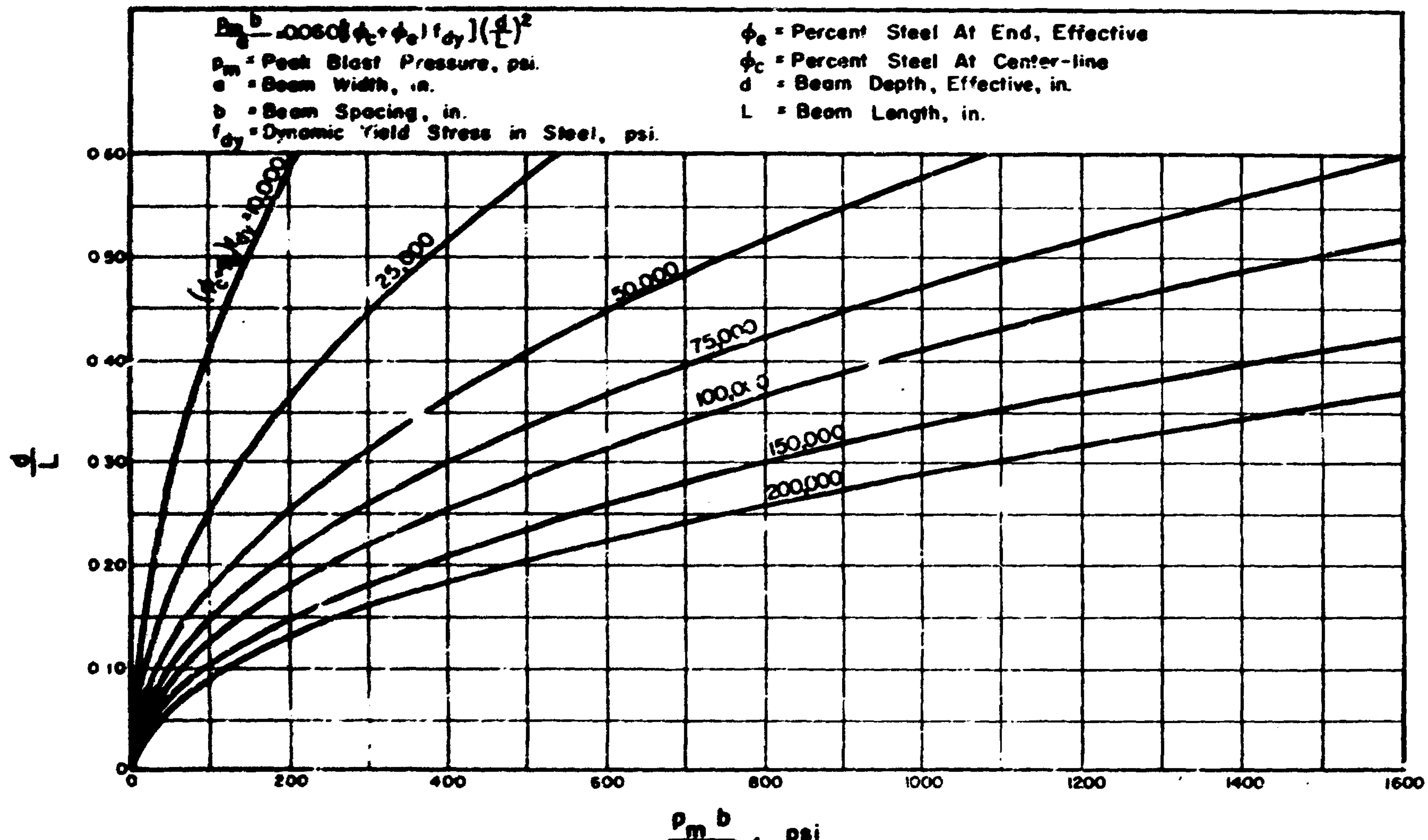


FIG. 5A-3.2 FLEXURAL RESISTANCE OF SIMPLY SUPPORTED OR CONTINUOUS REINFORCED CONCRETE BEAMS SUPPORTING ONE-WAY SLABS (μ = 3.0)

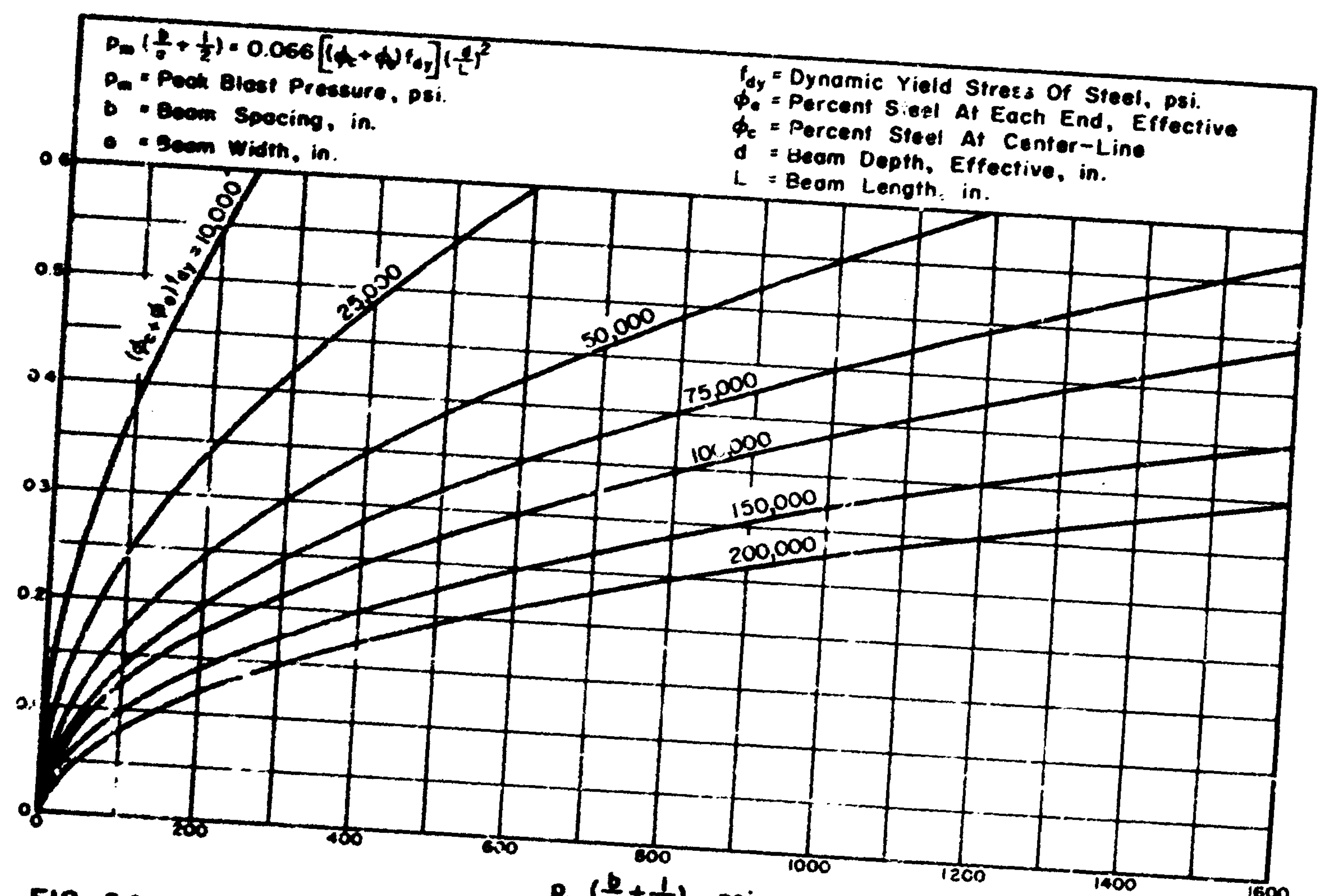


FIG. 5A-3.3 FLEXURAL RESISTANCE OF SIMPLY SUPPORTED OR CONTINUOUS REINFORCED CONCRETE BEAMS SUPPORTING SQUARE TWO-WAY SLABS ($u = 1.3$)

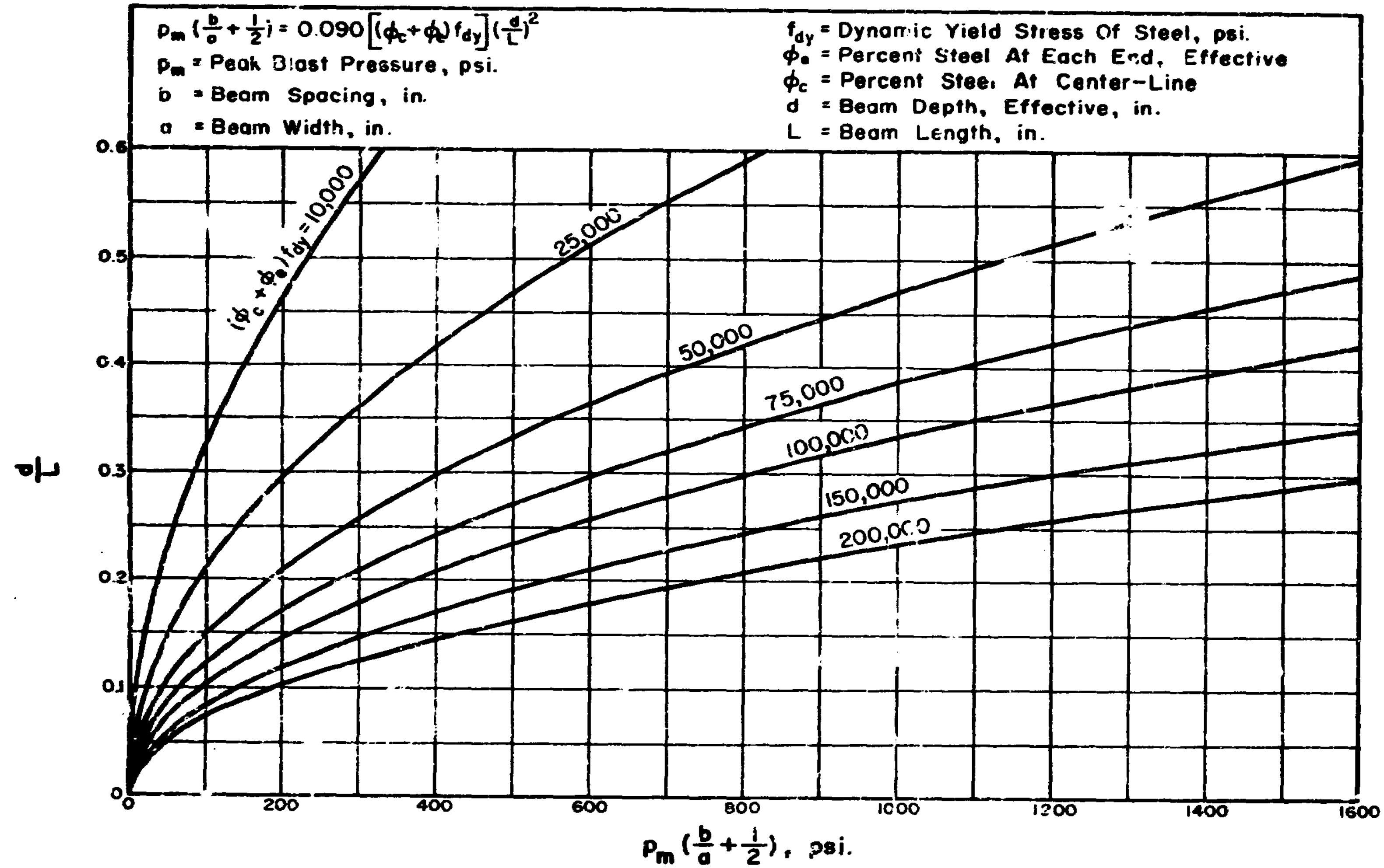
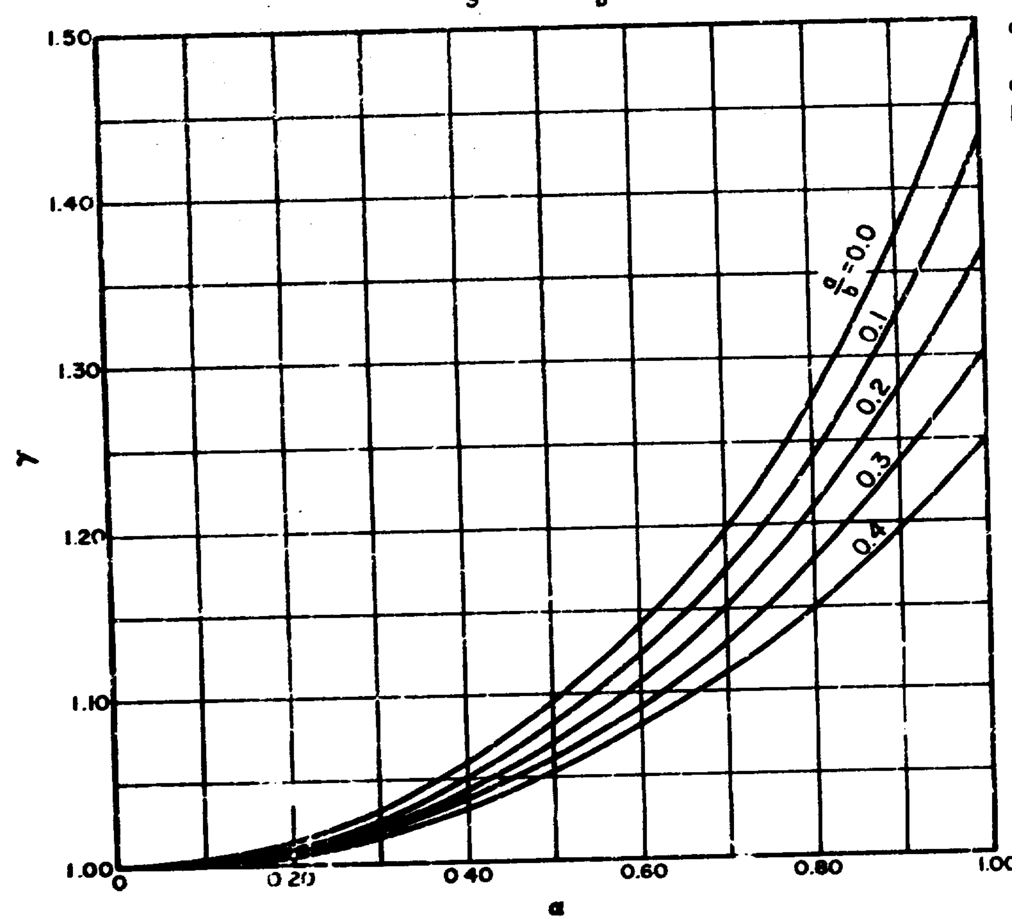


FIG. 5A-3.4 FLEXURAL RESISTANCE OF SIMPLY SUPPORTED OR CONTINUOUS REINFORCED CONCRETE BEAMS SUPPORTING SQUARE TWO-WAY SLABS (μ=3.0)

$$\gamma = \frac{1}{1-\frac{1}{3}\alpha^2\left(1-\frac{a}{b}\right)}$$



γ = Ratio Of Flexural Resistance Of Beam Supporting Two-Way Slab To That Of The Same Beam Supporting A One-Way Slab.

α = Ratio Of Short Side To Long Side Of Two-Way Slab.

a = Width Of Beam.

b = Spacing Center To Center Of Beams.

NOTES:

(A) To Obtain Resistance Of Beams Under Square Two-Way Slabs And Of Long Beam Under Non-Square Slabs:

(1) Treat As Beam Under A One-Way Slab. Get Resistance From Figs. 5A-3.1,-3.2 -4.1, or -4.2.

(2) Multiply Value From (1) Above By γ.

(B) To Obtain Resistance Of Short Beams Under Non-Square Two-Way Slab:

(1) Treat As Beam Under A Square Two-Way Slab Of Span Equal To That Of Short Beam Being Considered.

FIG. 5A-3.5 FLEXURAL RESISTANCE — FACTOR FOR BEAMS SUPPORTING NON-SQUARE TWO-WAY SLABS.

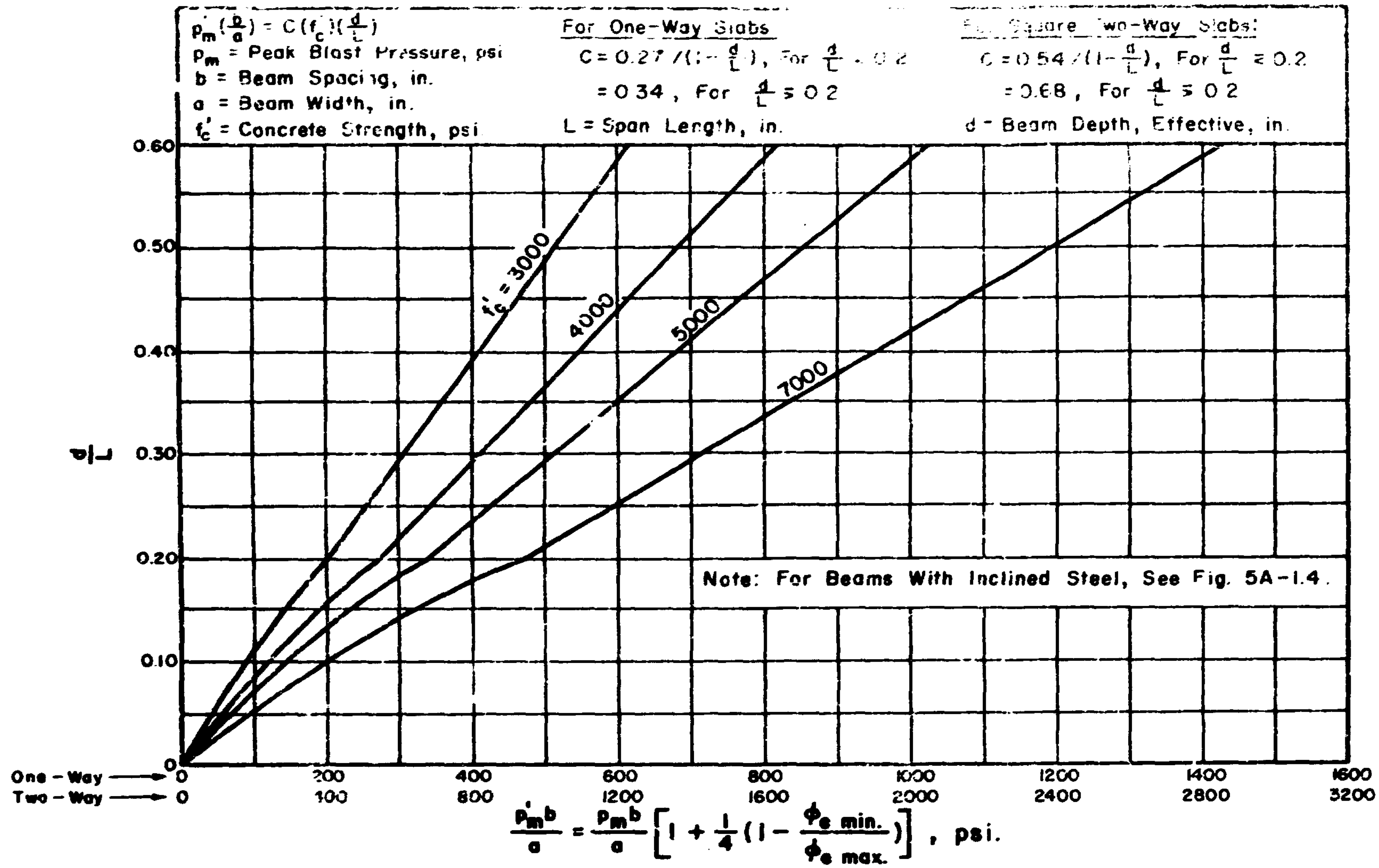


FIG. 5A-3.6 RESISTANCE IN PURE SHEAR OF REINFORCED CONCRETE BEAMS SUPPORTING ONE-WAY AND SQUARE TWO-WAY SLABS (μ = 1.3).

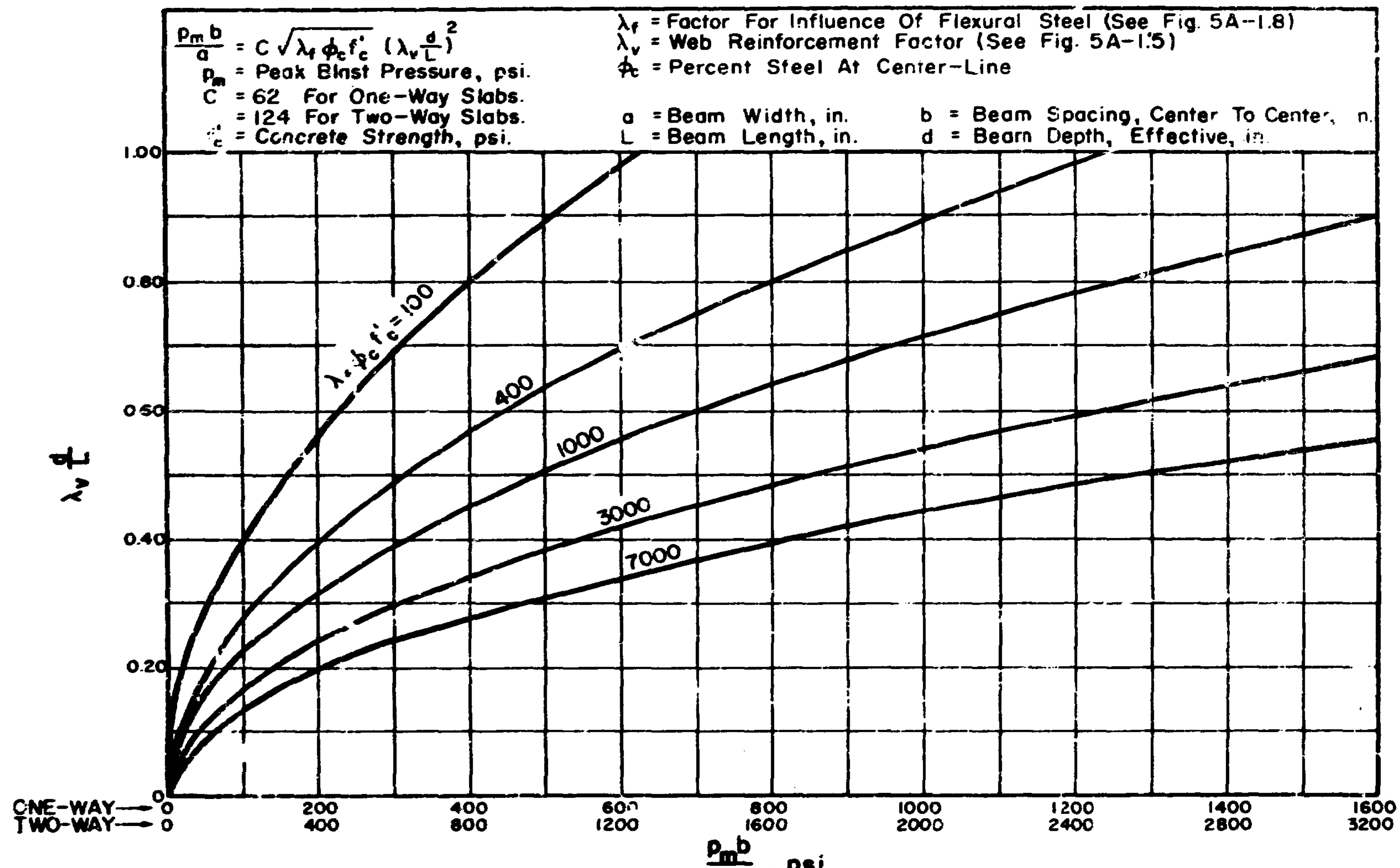


FIG. 5A-3.7 RESISTANCE IN DIAGONAL TENSION OF REINFORCED CONCRETE BEAMS SUPPORTING ONE-WAY AND SQUARE TWO-WAY SLABS (μ=1.3)

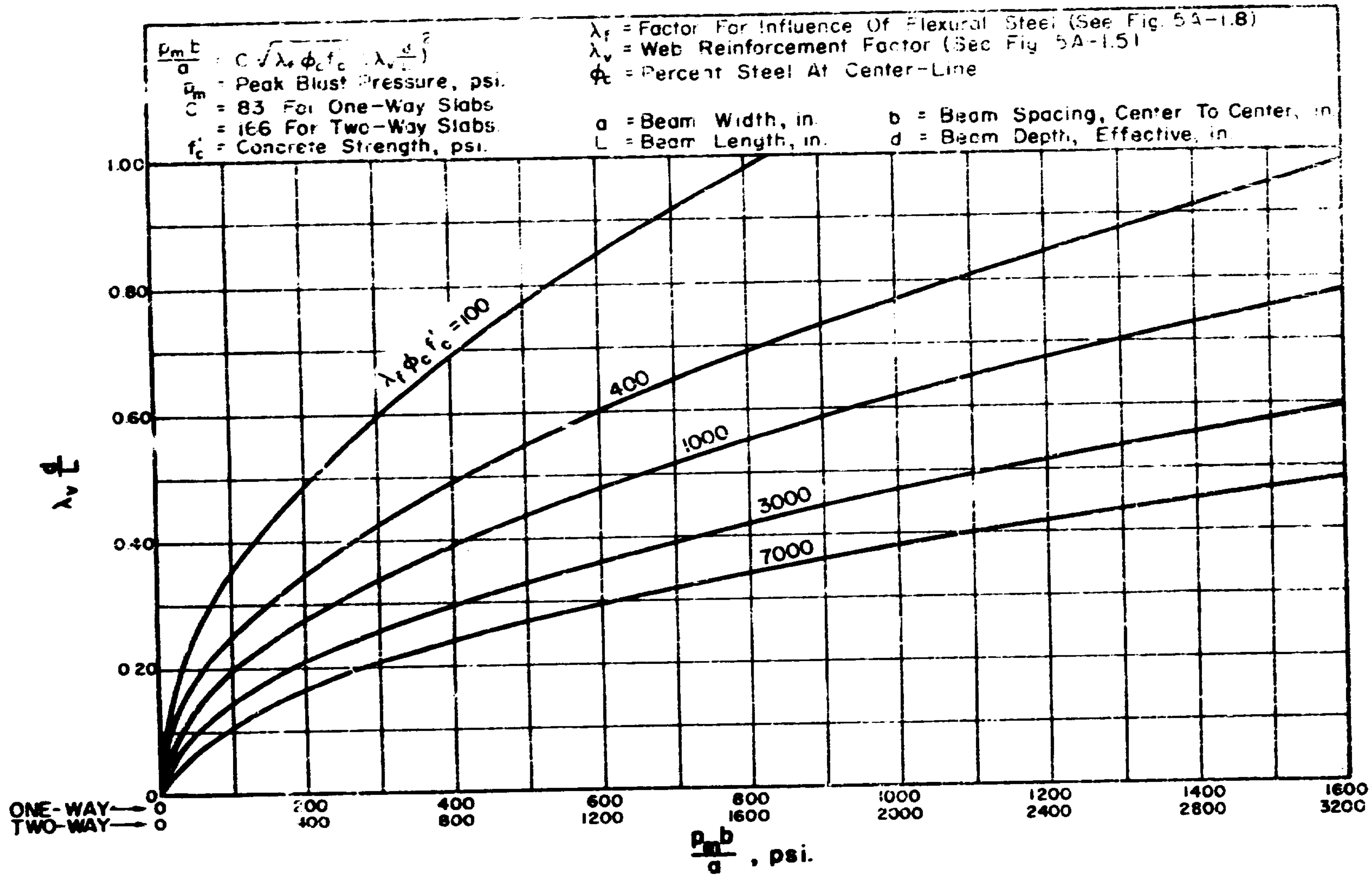
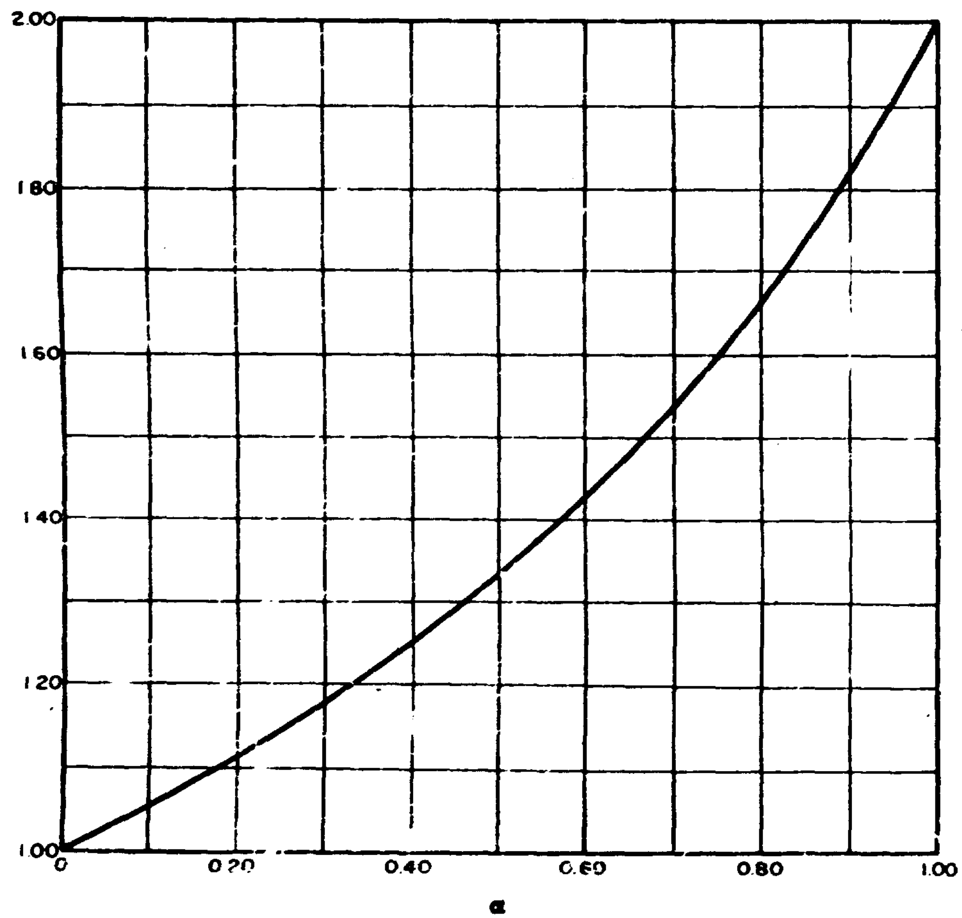


FIG. 5A-3.8 RESISTANCE IN DIAGONAL TENSION OF REINFORCED CONCRETE BEAMS SUPPORTING ONE-WAY AND SQUARE TWO-WAY SLABS ($\mu = 3.0$)



$\eta = \frac{2}{2-a}$

η = Correction Factor For Resistance In Shear Or Diagonal Tension Of Long Beam Under Two-Way Slab

a = Ratio Of Short Side To Long Side Of Two-Way Slab

NOTE:

To Obtain Resistance In Shear And/Or Diagonal Tension For Long Beam Under Two-Way Slab:

(1) Treat As Beam Under One-Way Slab. Get Resistance From Figures 5A-3.6,-3.7,-3.8; ;4.5, & -4.6

(2) Multiply Value From (1) Above By η

Resistance Of Short Span Beam Is Same As For Beam Under a Square Slab Of That Span. (See Figure 5A-3.6,-3.7, -3.8, -4.5, and -4.6.

FIG. 5A-3.9 RESISTANCE IN SHEAR AND DIAGONAL TENSION — FACTOR FOR BEAMS SUPPORTING NON-SQUARE TWO-WAY SLABS

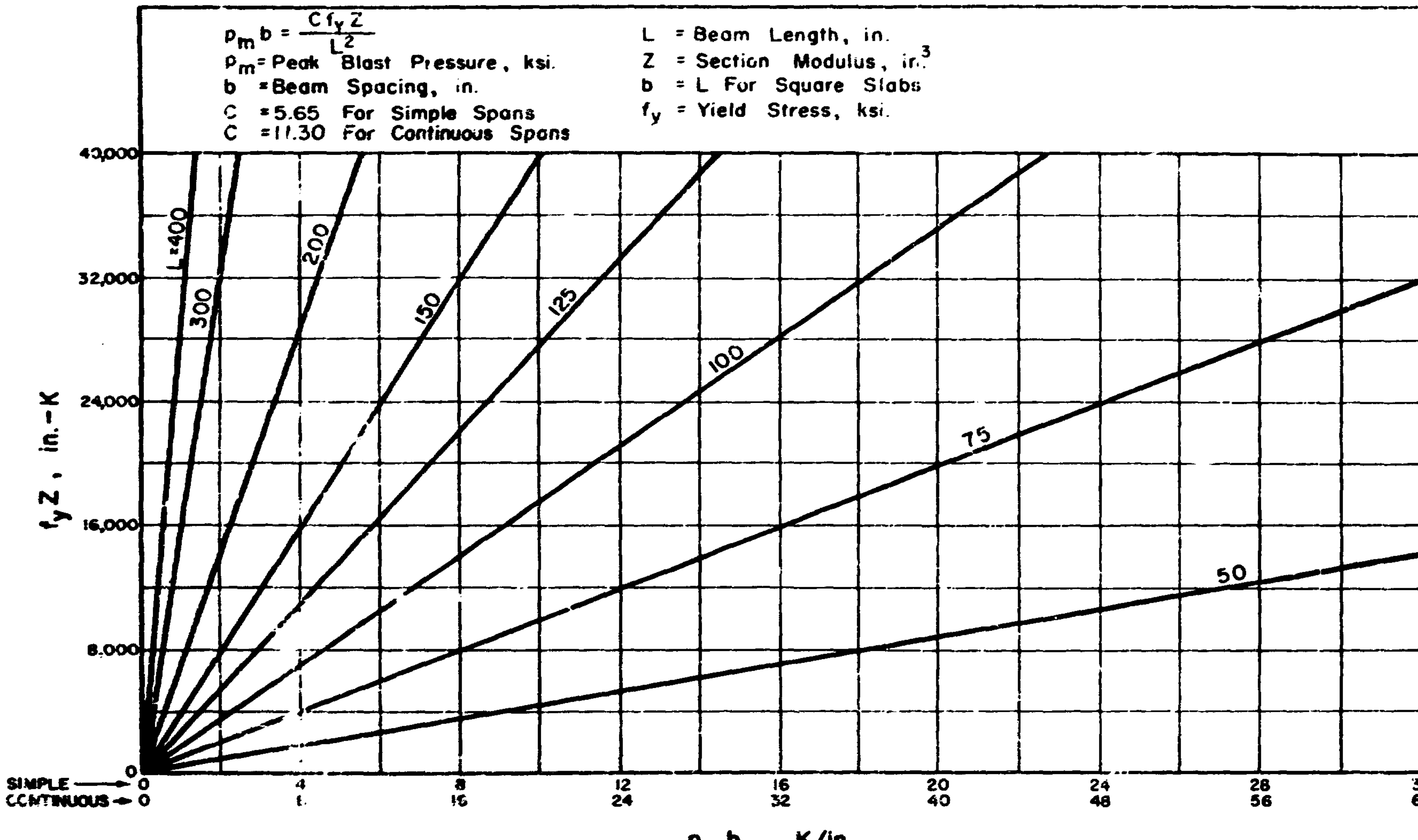


FIG. 5A-4.1 FLEXURAL RESISTANCE OF STEEL BEAMS SUPPORTING ONE-WAY SLABS (μ = 1.3)

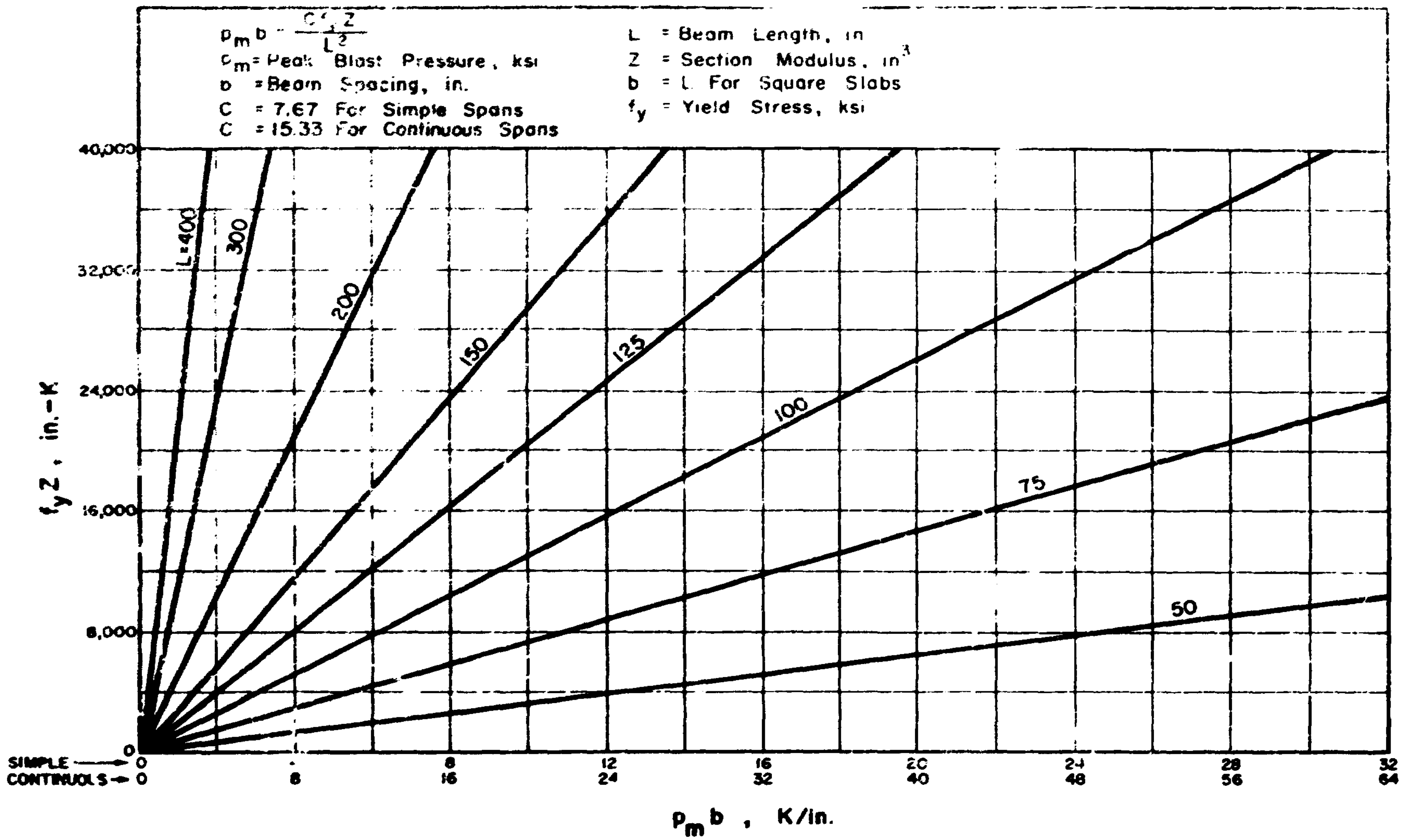


FIG. 5A-4.2 FLEXURAL RESISTANCE OF STEEL BEAMS SUPPORTING ONE-WAY SLABS (μ = 3.0)

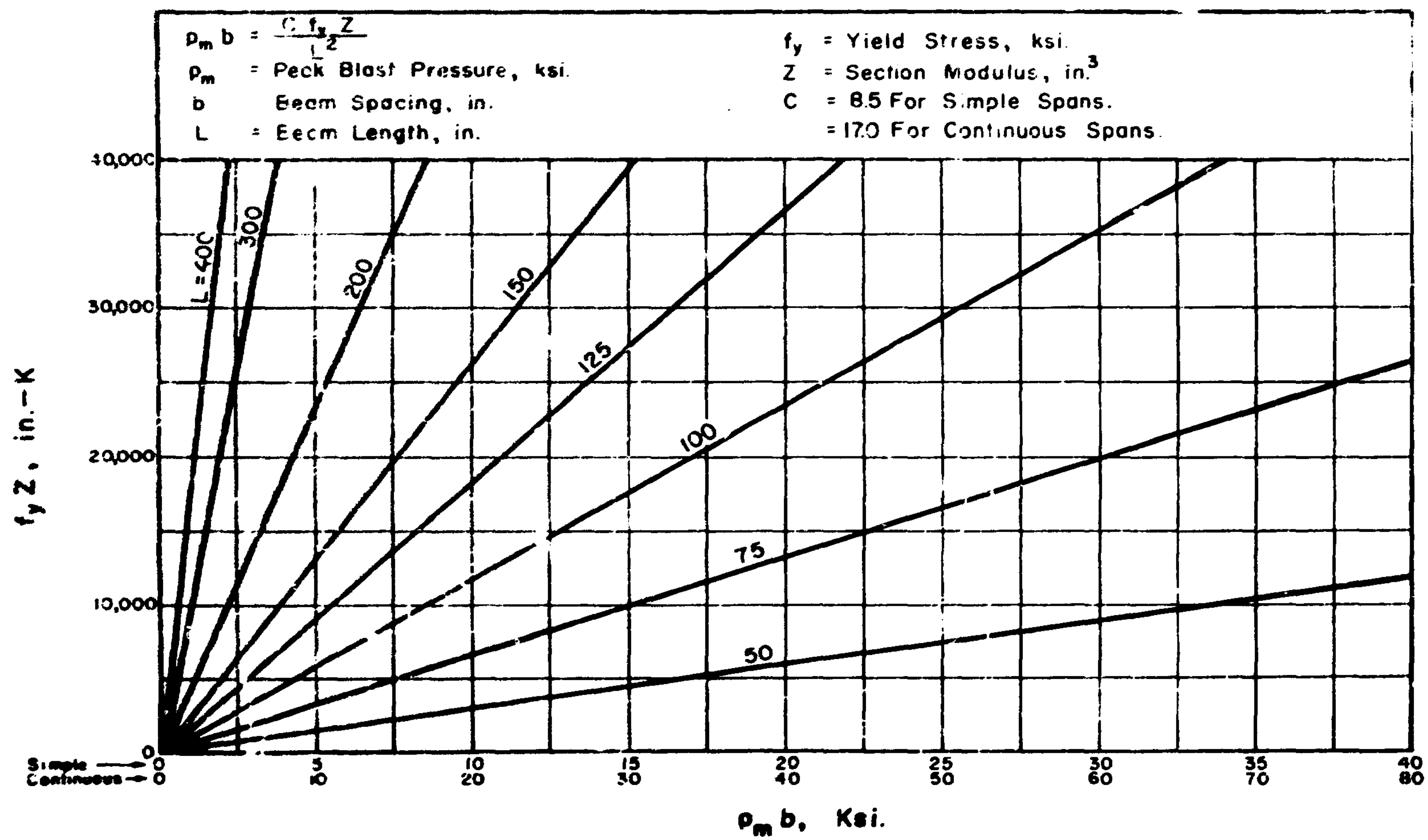


FIG. 5A-4.3 FLEXURAL RESISTANCE OF STEEL BEAMS SUPPORTING SQUARE TWO-WAY SLABS (μ = 1.3)

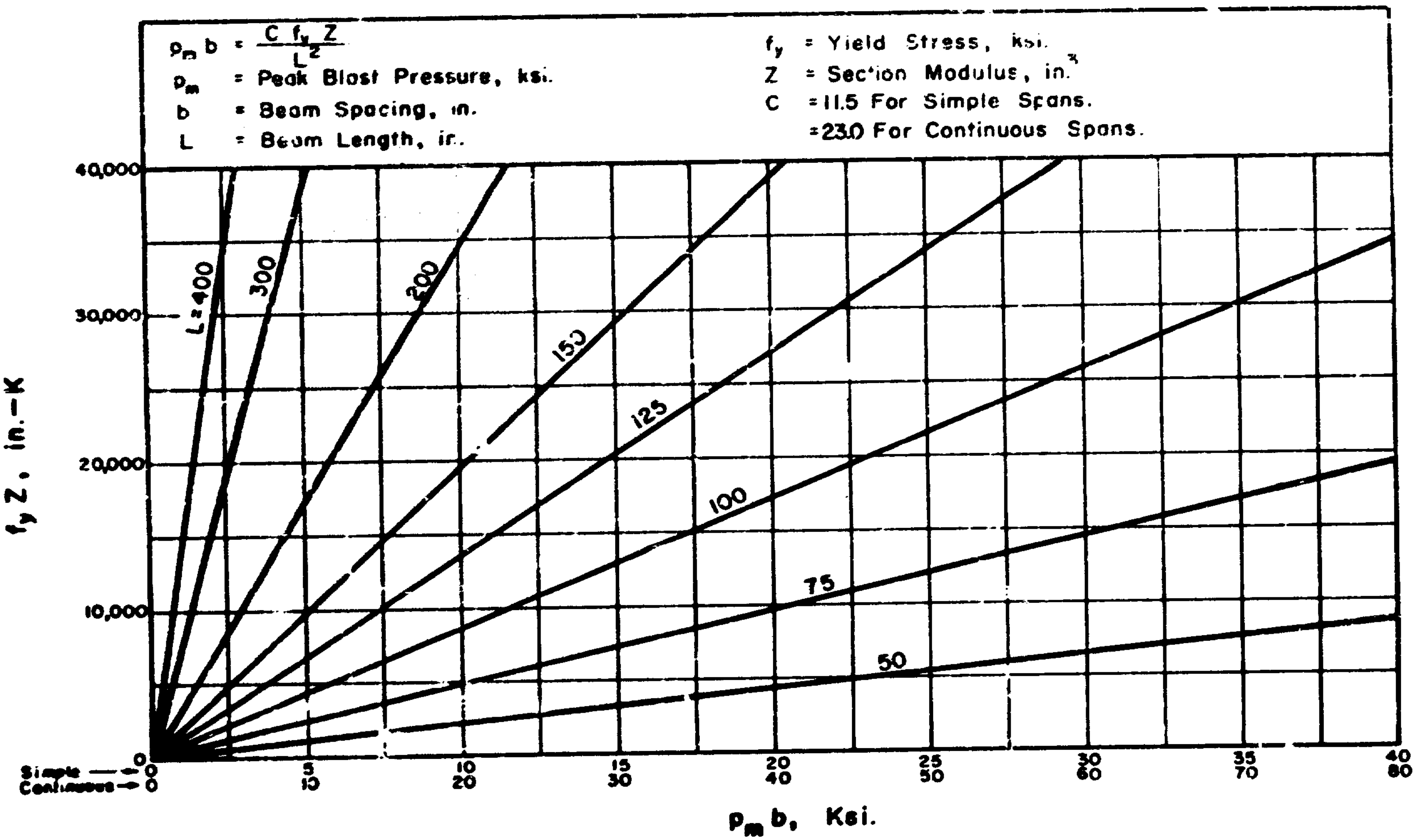


FIG. 5A-4.4 FLEXURAL RESISTANCE OF STEEL BEAMS SUPPORTING SQUARE TWO-WAY SLABS ($\mu = 3.0$)

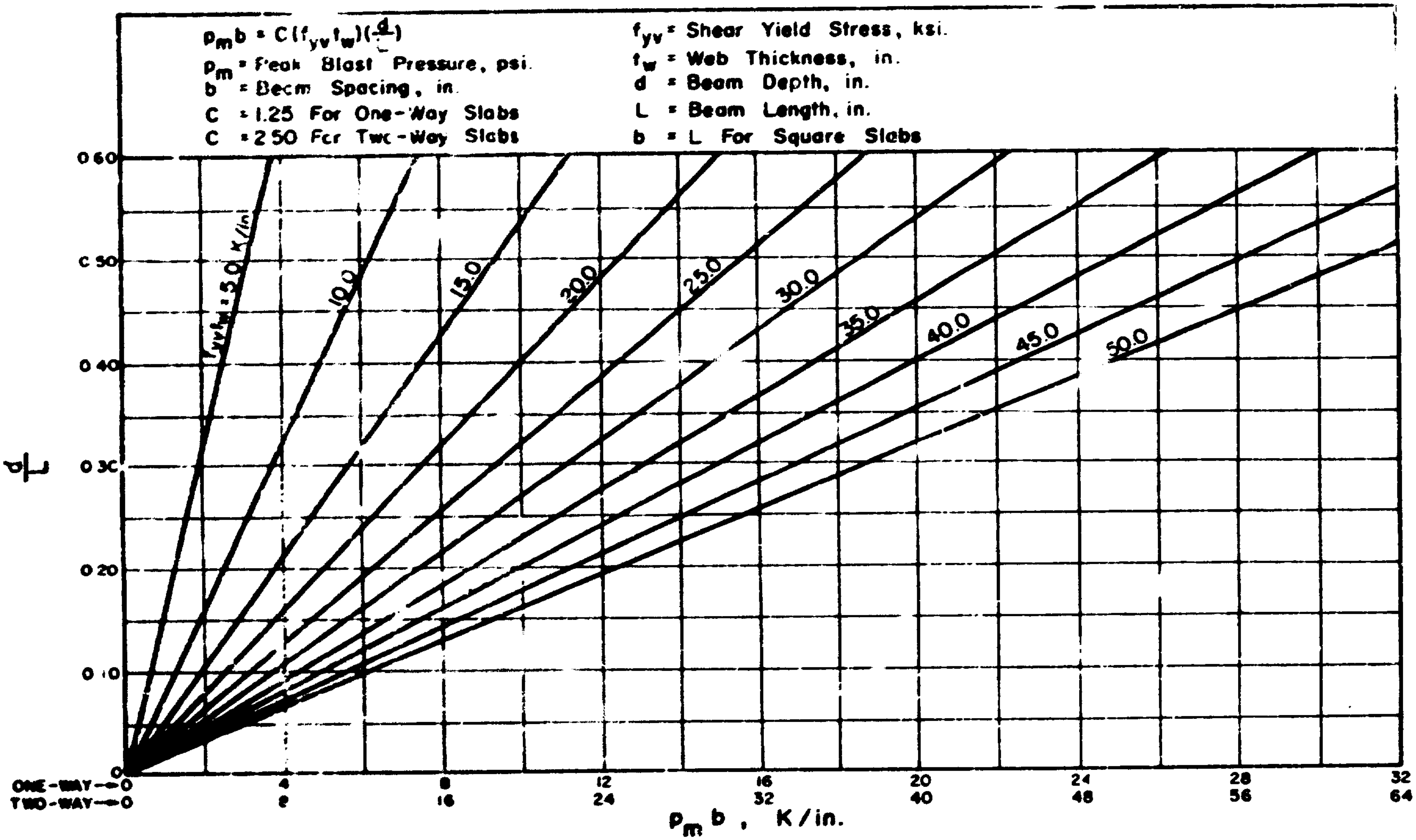


FIG. 5A-4.5 SHEARING RESISTANCE OF SIMPLE AND CONTINUOUS STEEL BEAMS SUPPORTING ONE-WAY AND SQUARE TWO-WAY SLABS (μ = 1.3)

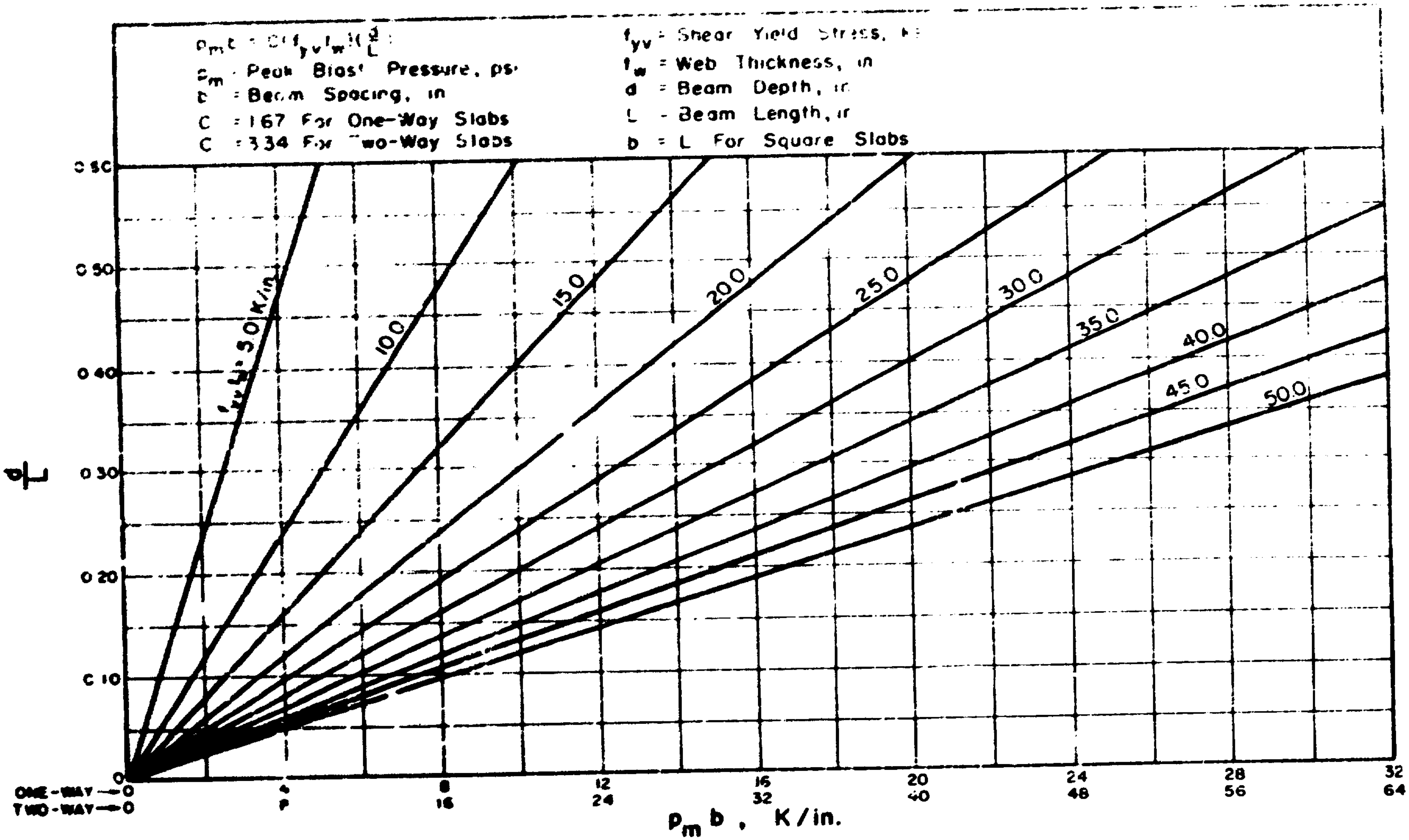


FIG. 5A-4.6 SHEARING RESISTANCE OF SIMPLE AND CONTINUOUS STEEL BEAMS SUPPORTING ONE-WAY AND SQUARE TWO-WAY SLABS (μ = 3.0)

$P_{mf}K = 0.0443 f_{dy} \phi \left(\frac{d}{L_1}\right)^2$ When $\frac{L_2}{L_1} = \frac{p_2}{p_1} = \frac{c_2}{c_1}$

P_{mf} = Peak Blast Pressure Corresponding To Flexural Strength, psi

K = A Function Of L_2/L_1 And c_1/L_1, See Fig. 5A-54

d and dp = Effective Depth Of Slab And Drop Panel Respectively, in.

L_1, c_1, and p_1 = Long Side Of Flat Slab Panel, Column Capital, And Drop Panel Respectively, in.

L_2, c_2, and p_2 = Short Side Of Flat Slab Panel, Column Capital, And Drop Panel Respectively, in.

$\phi = \phi_{b1} + \phi_{b2} + X(\phi_{t1} + \phi_{t2}) + X'(\phi'_{t1} + \phi'_{t2})$ where X And X' Are As Shown In Figure 5A-5.3

f_{dy} = Dynamic Steel Yield Strength, psi.

ϕ_{b1} and ϕ_{b2} = Average Steel Percentage At Bottom Parallel To Sides L_2 And L_1 Respectively.

ϕ_{t1} and ϕ_{t2} = Average Steel Percentage Between Drop Panels At Top Parallel To Sides L_2 And L_1 Respectively.

ϕ'_{t1} and ϕ'_{t2} = Average Steel Percentage Over Drop Panels At Top Parallel To Sides L_2 And L_1 Respectively.

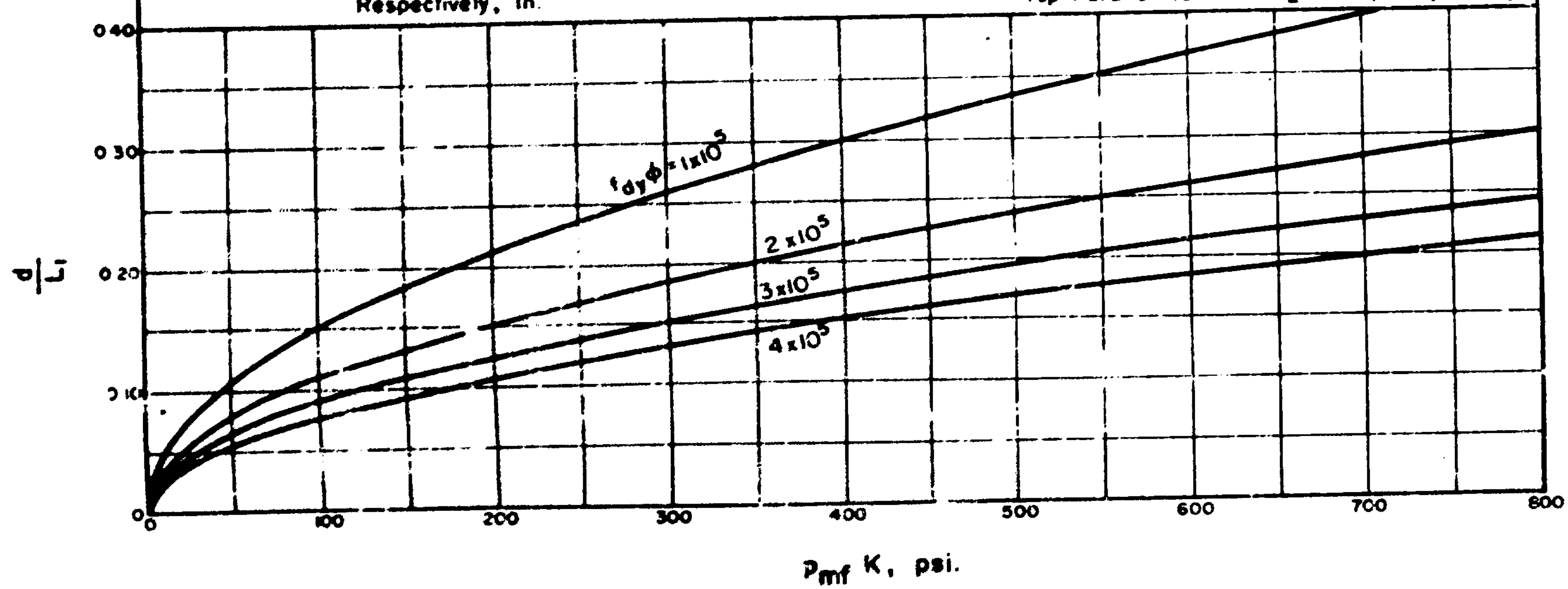


FIG. 5A-5.1 FLEXURAL RESISTANCE OF FLAT SLABS (μ = 1.3)

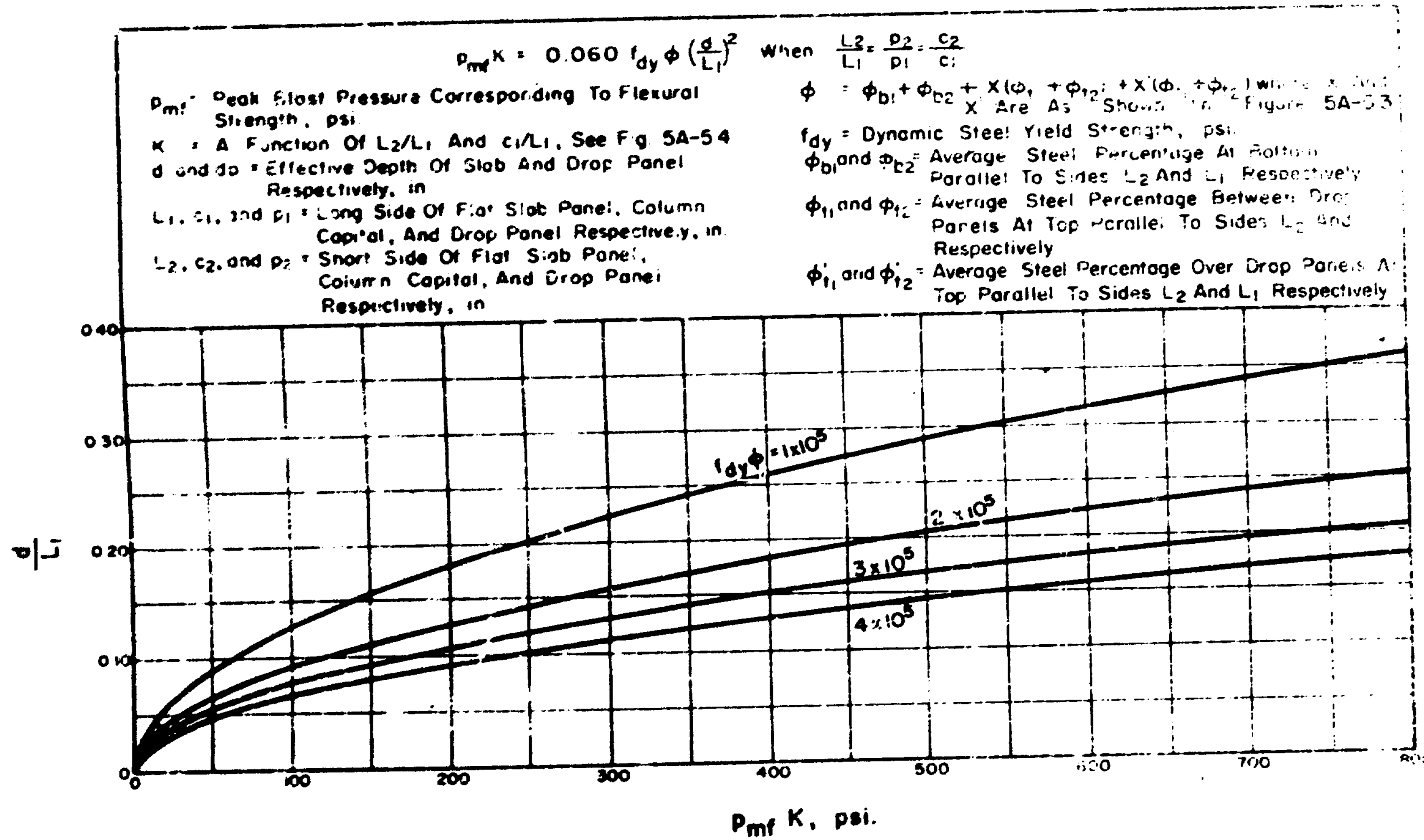


FIG. 5A-5.2 FLEXURAL RESISTANCE OF FLAT SLABS (μ = 3.0)

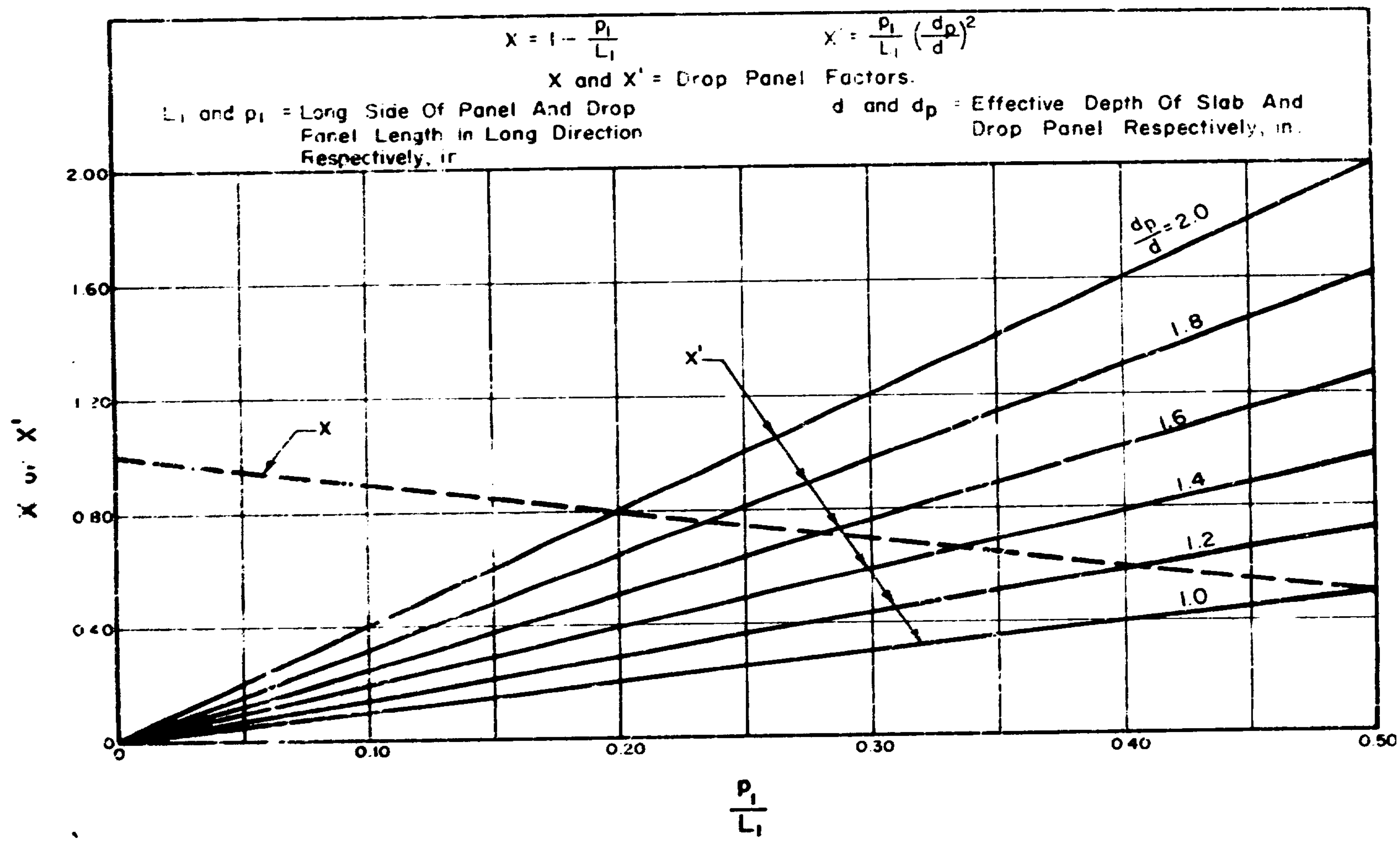


FIG. 5A-5.3 DROP PANEL FACTORS FOR FLAT SLABS

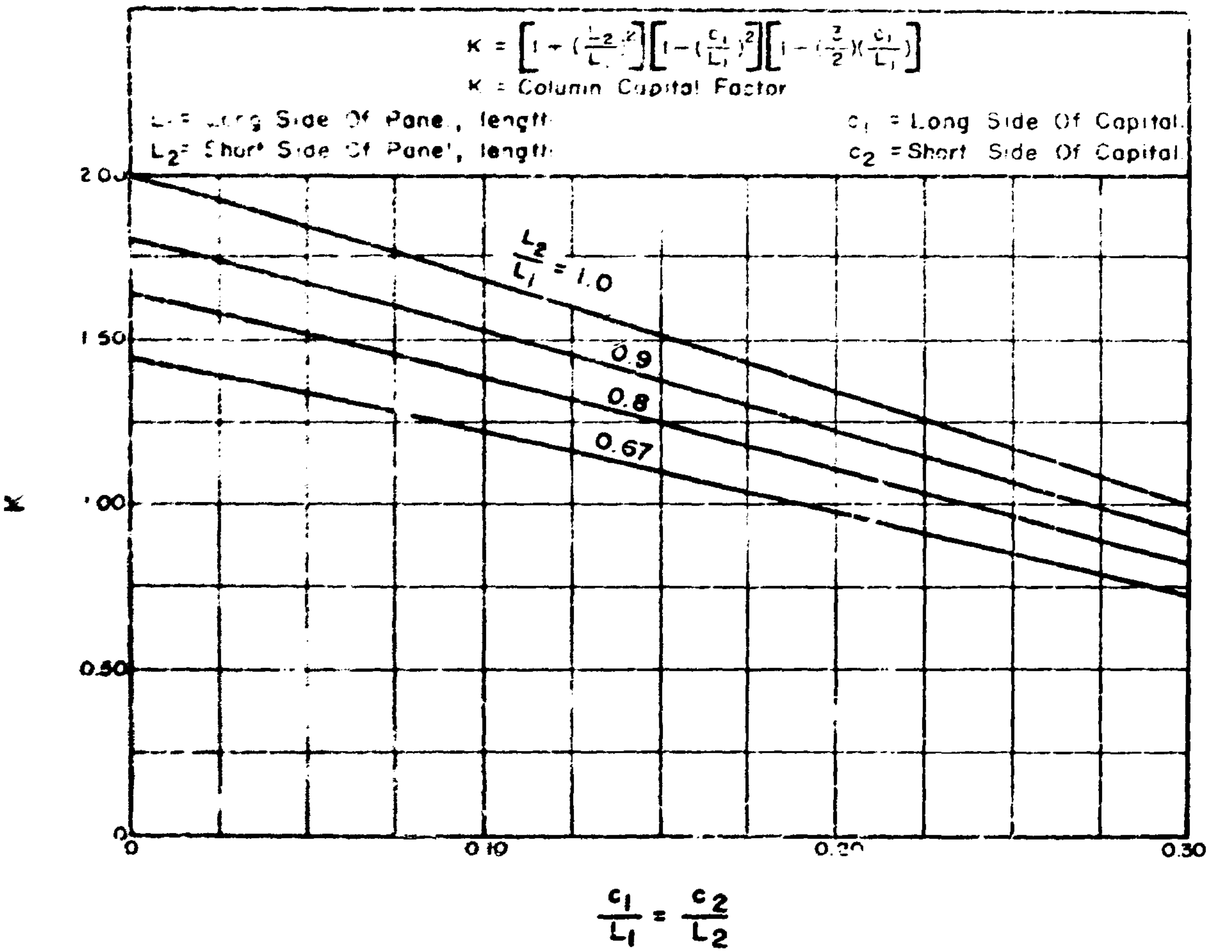


FIG. 5A-5.4 COLUMN CAPITAL FACTOR FOR FLEXURAL STRENGTH OF FLAT SLABS.

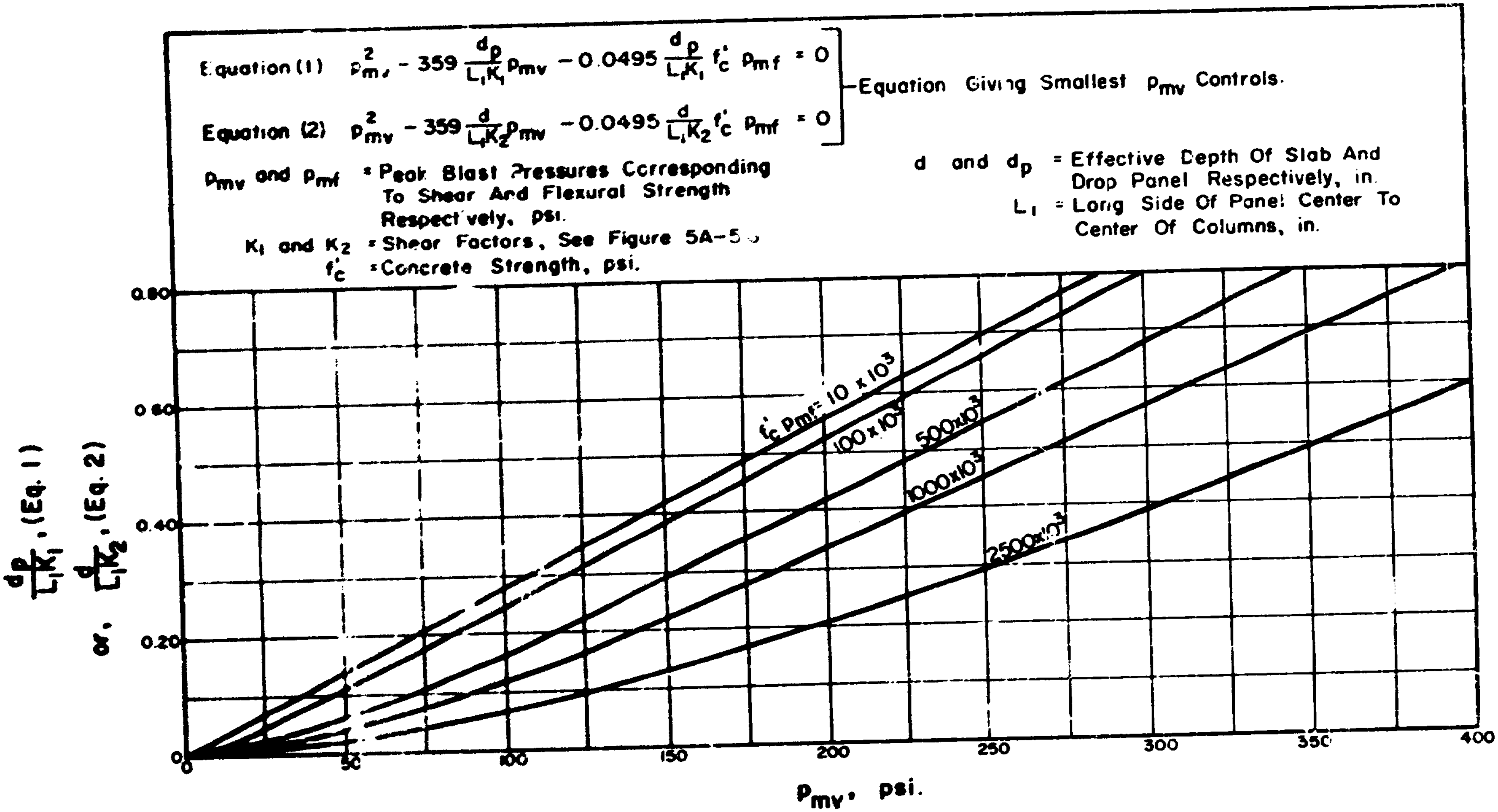
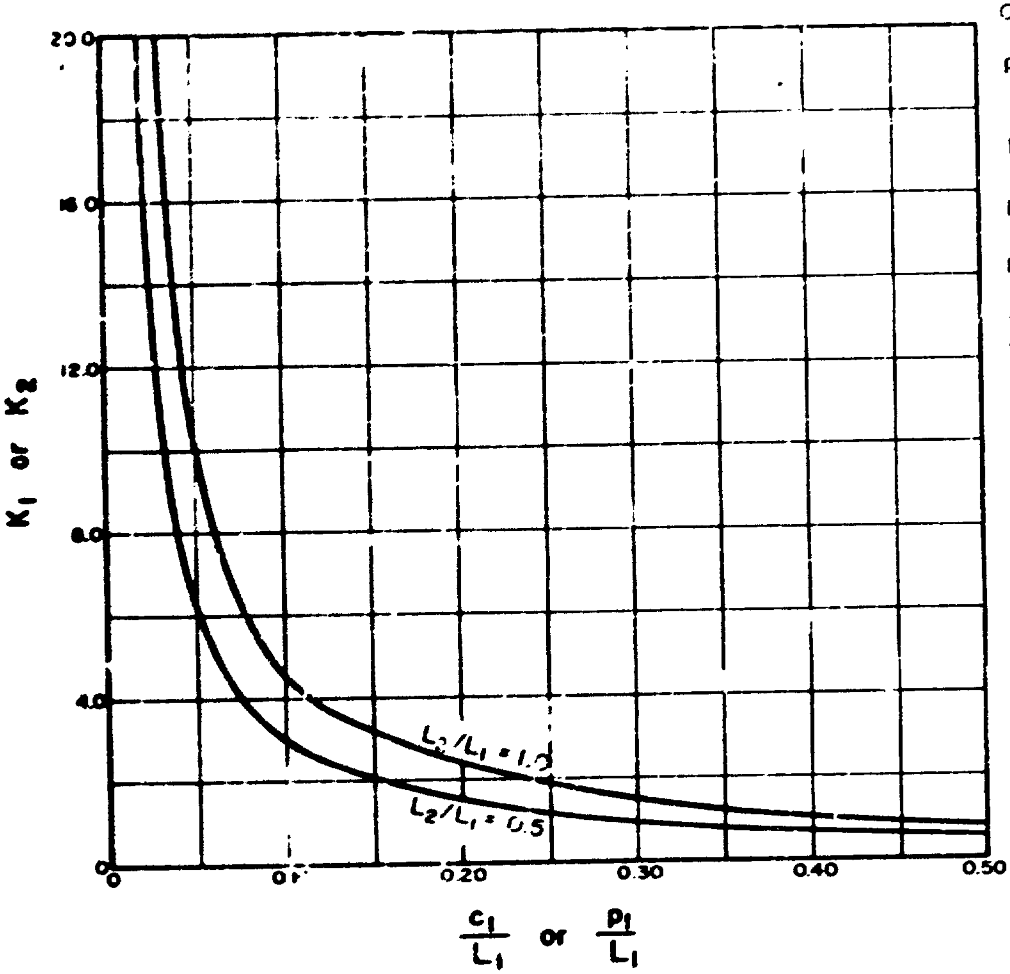


FIG. 5A-5.5 SHEAR STRENGTH OF FLAT SLABS (μ = 1.3)



Calculate p_{mv} By Using Smallest p_{mv} Of Following:

$$p_{mv}^2 - \frac{583}{K_1}\left(\frac{2\mu-1}{2\mu}\right)\left(\frac{d_p}{L_1}\right)p_{mv} - \frac{0.0805}{K_1} f'_c \left(\frac{2\mu-1}{2\mu}\right)\left(\frac{d_p}{L_1}\right) p_{mf}$$

$$p_{mv}^2 - \frac{583}{K_2}\left(\frac{2\mu-1}{2\mu}\right)\left(\frac{d}{L_1}\right)p_{mv} - \frac{0.0805}{K_2} f'_c \left(\frac{2\mu-1}{2\mu}\right)\left(\frac{d}{L_1}\right) p_{mf}$$

p_{mv} = Peak Blast Press. Corresponding To Shear Strength, psi.

p_{mf} = Peak Blast Pressure Corresponding To Flexural Strength, psi. (See Figure

d = Effective Depth Of Slab, in.

d_p = Effective Depth Of Drop Panel, in.

f'_c = Concrete Strength, psi.

L_1, c_1, and p_1 = Long Side Of Flat Slab Panel, Column Capital, And Drop Panel Respectively, in.

L_2, c_2, and p_2 = Short Side Of Flat Slab Panel, Column Capital, And Drop Panel Respectively, in.

$$K_1 = \frac{\frac{L_1}{c_1} - \frac{c_1}{L_1}}{\frac{L_1}{L_2} + 1}, \quad K_2 = \frac{\frac{L_1}{p_1} - \frac{p_1}{L_1}}{\frac{L_1}{L_2} + 1}$$

$$\frac{L_2}{L_1} = \frac{c_2}{c_1} = \frac{p_2}{p_1}$$

μ = Ductility Factor = Ratio Of Maximum Deflection To Yield Deflection.

FIG. 5A-5.6 SHEAR FACTORS FOR FLAT SLABS

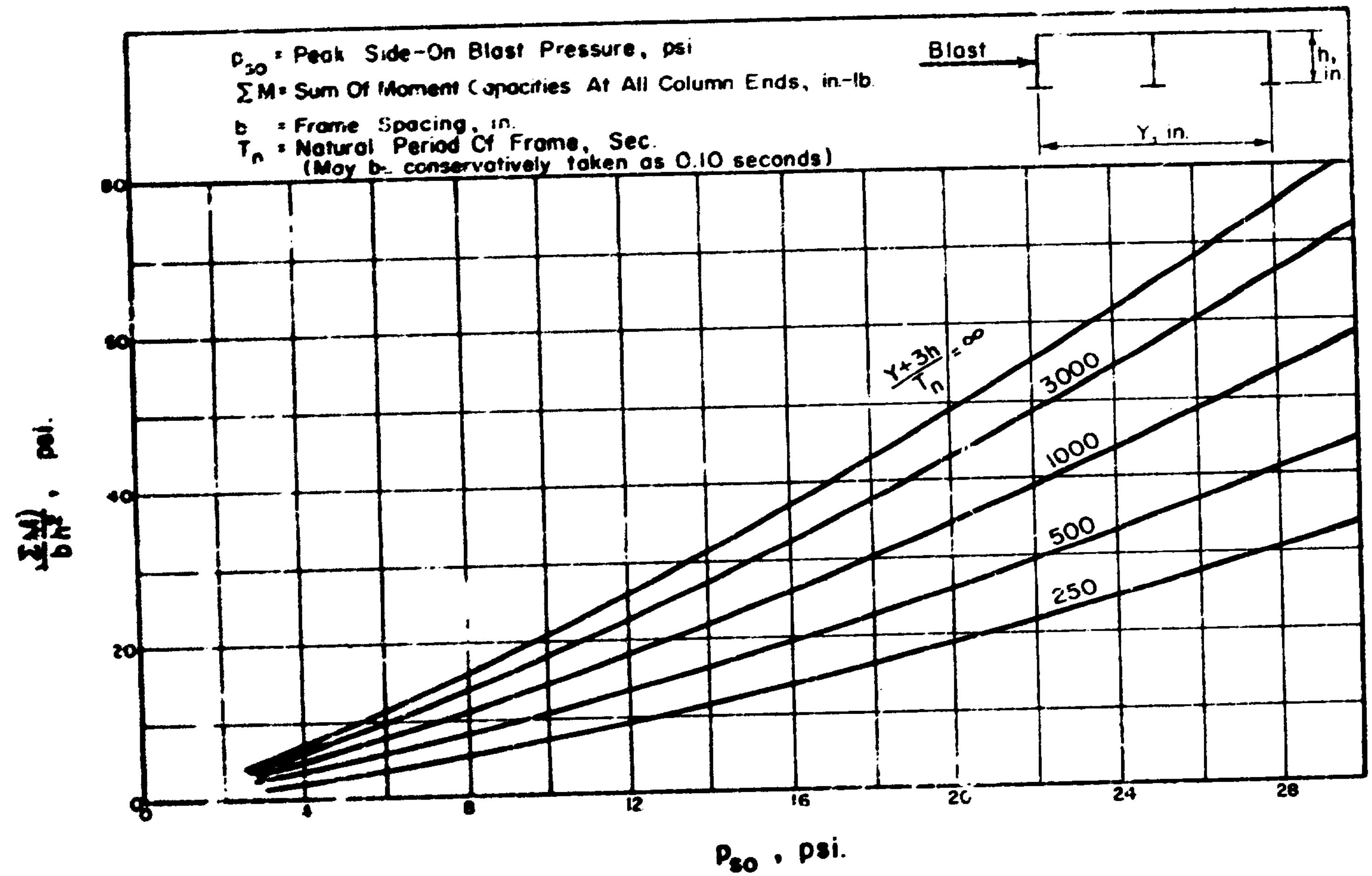


FIG. 5A-6.1 RESISTANCE OF ABOVE-GROUND ONE-STORY RIGID FRAMES — WINDOW-LESS BUILDINGS ($\mu = 1.3$)

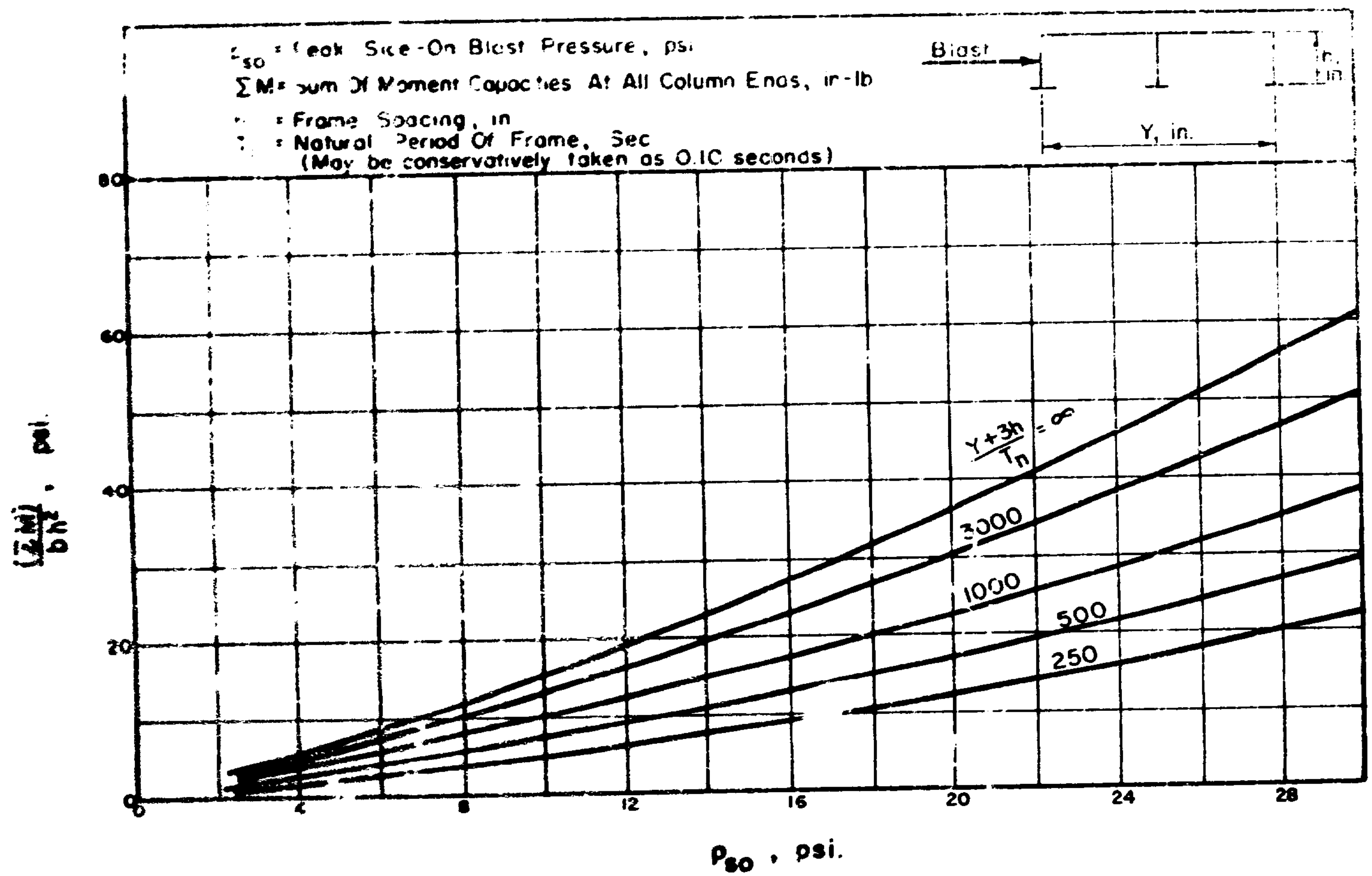


FIG. 5A-6.2 RESISTANCE OF ABOVE-GROUND ONE-STORY RIGID FRAMES — WINDOW-LESS BUILDINGS (μ = 3.0)

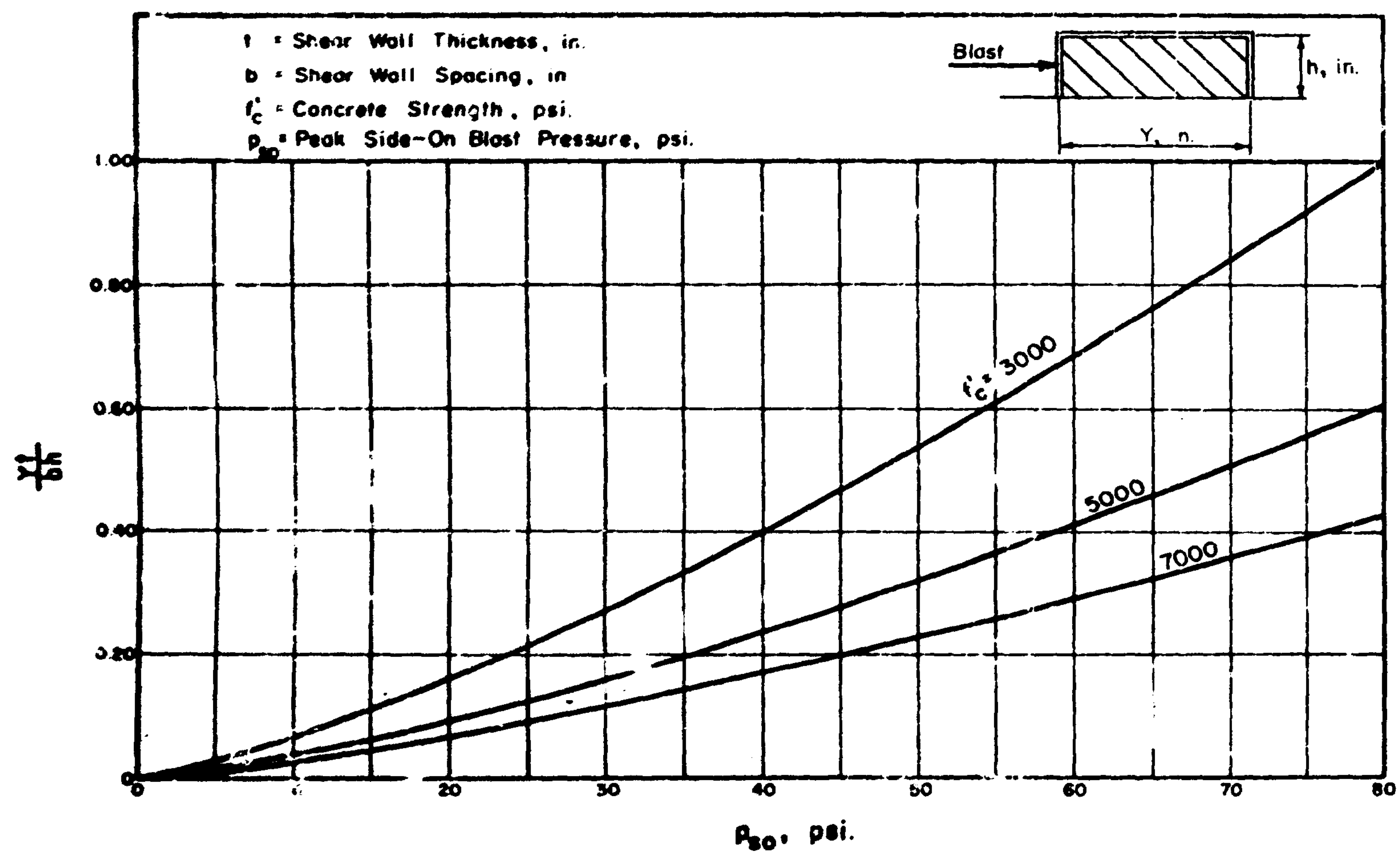


FIG. 5A-6.3 RESISTANCE OF ABOVE-GROUND ONE-STORY SHEAR WALLS — WINDOWLESS BUILDINGS (μ = 1.3)

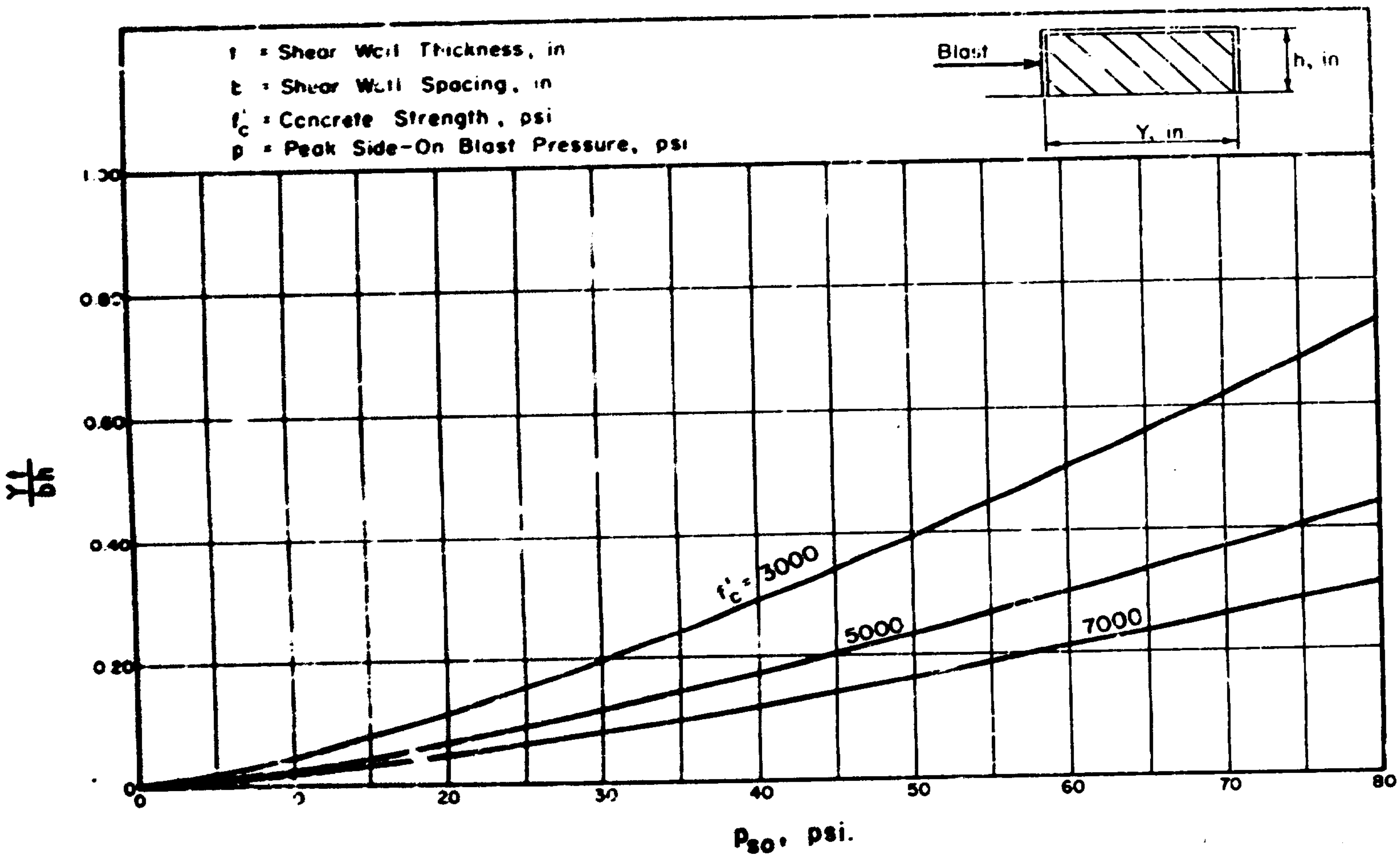


FIG. 5A-6.4 RESISTANCE OF ABOVE-GROUND ONE-STORY SHEAR WALLS —— WINDOW-LESS BUILDINGS (μ = 3.0)

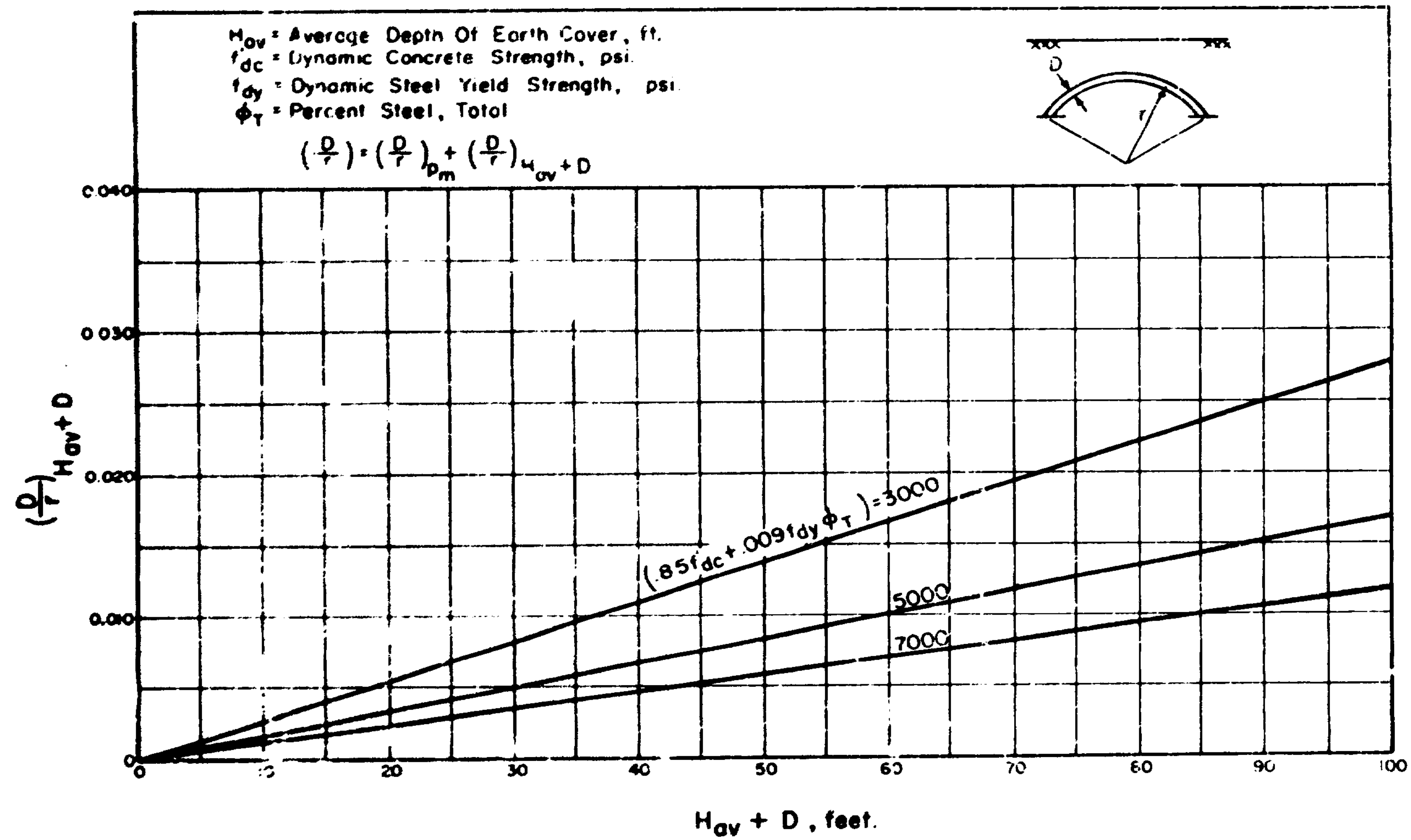


FIG. 5A-71 REQUIRED THICKNESS OF FULLY-BURIED REINFORCED CONCRETE ARCHES FOR DEAD LOAD

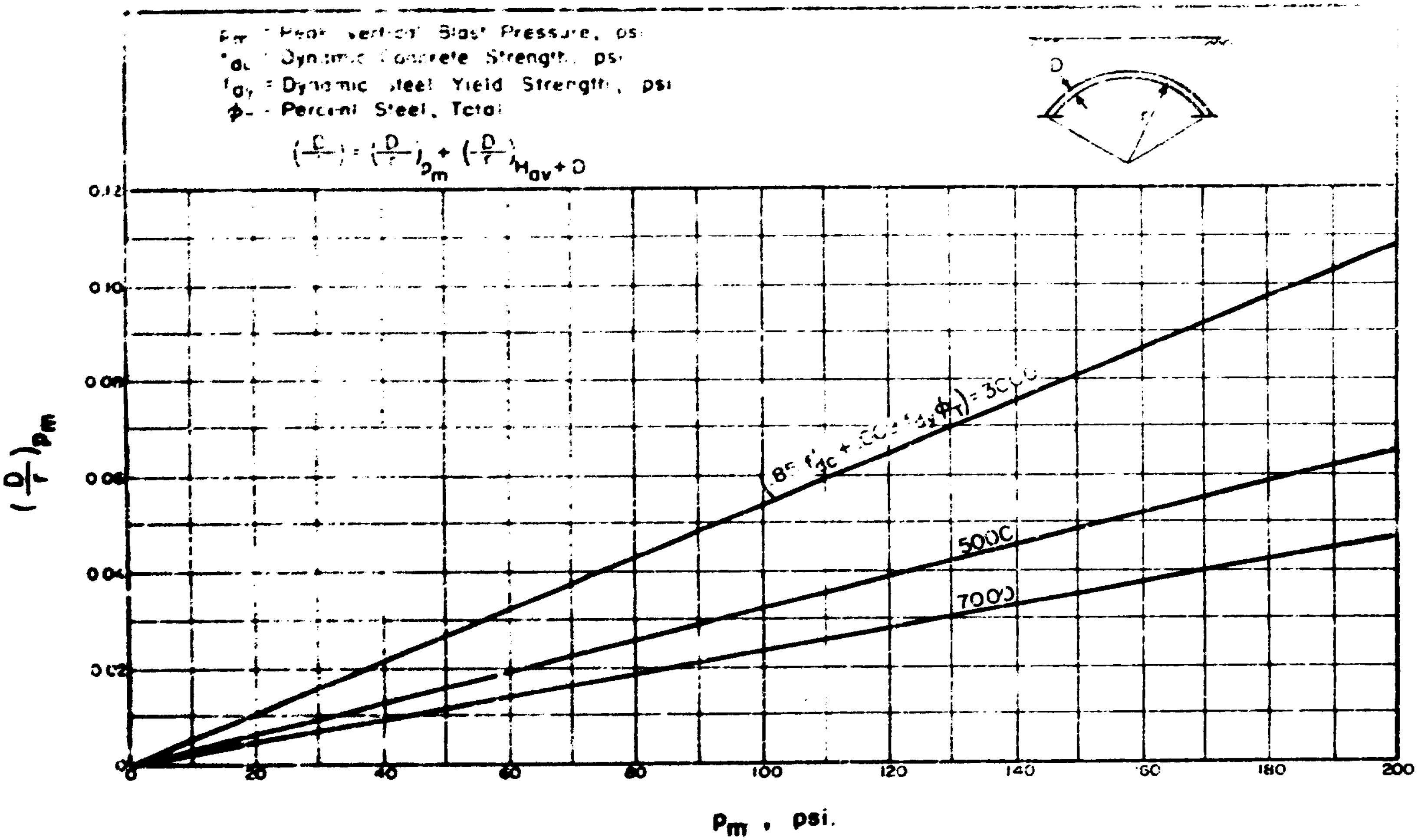


FIG. 5A-7.2 REQUIRED THICKNESS OF FULLY-BURIED REINFORCED CONCRETE ARCHES FOR BLAST LOAD (μ = 1.3)

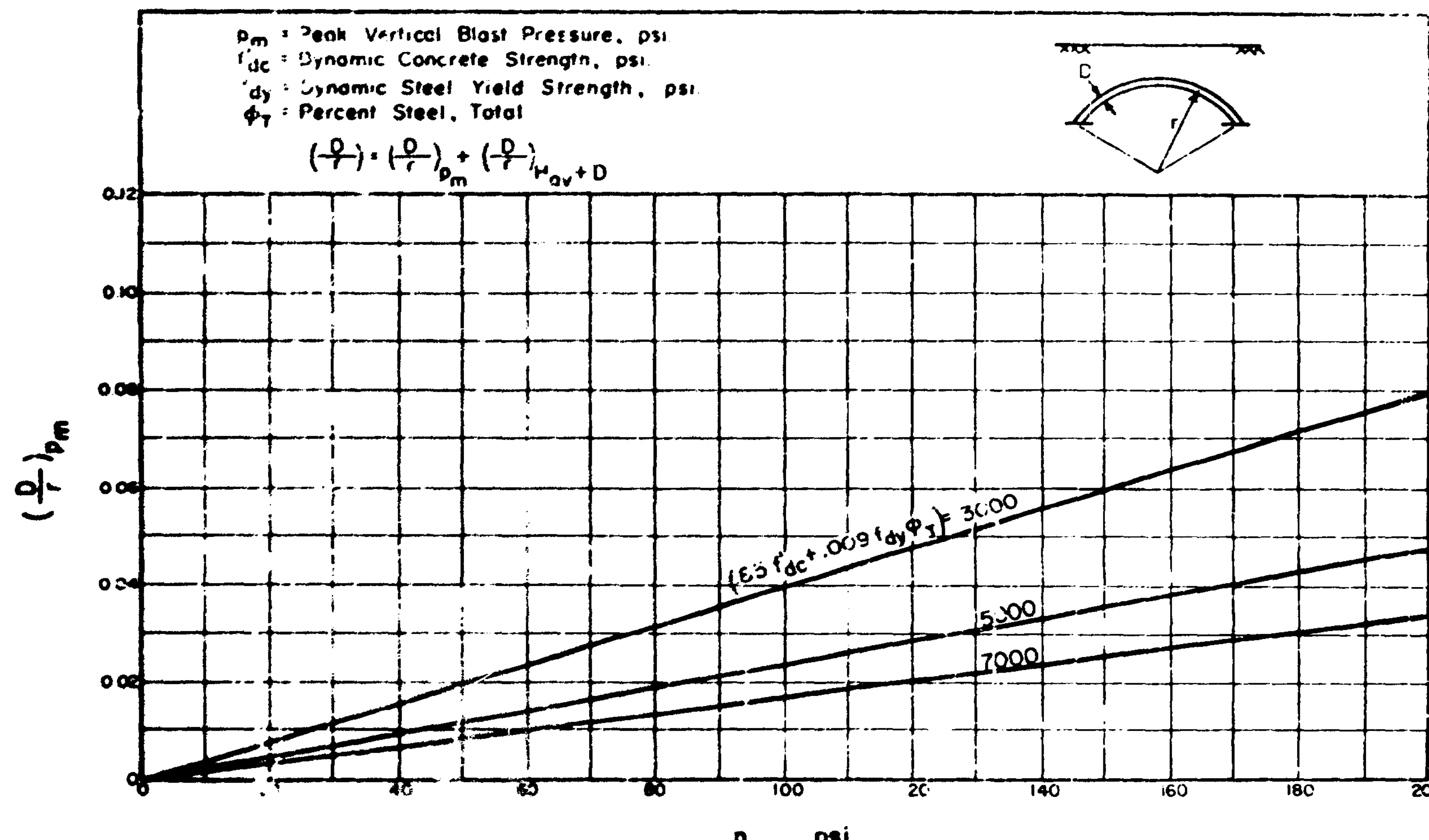


FIG. 5A-7.3 REQUIRED THICKNESS OF FULLY-BURIED REINFORCED CONCRETE ARCHES FOR BLAST LOAD (μ = 3.0)

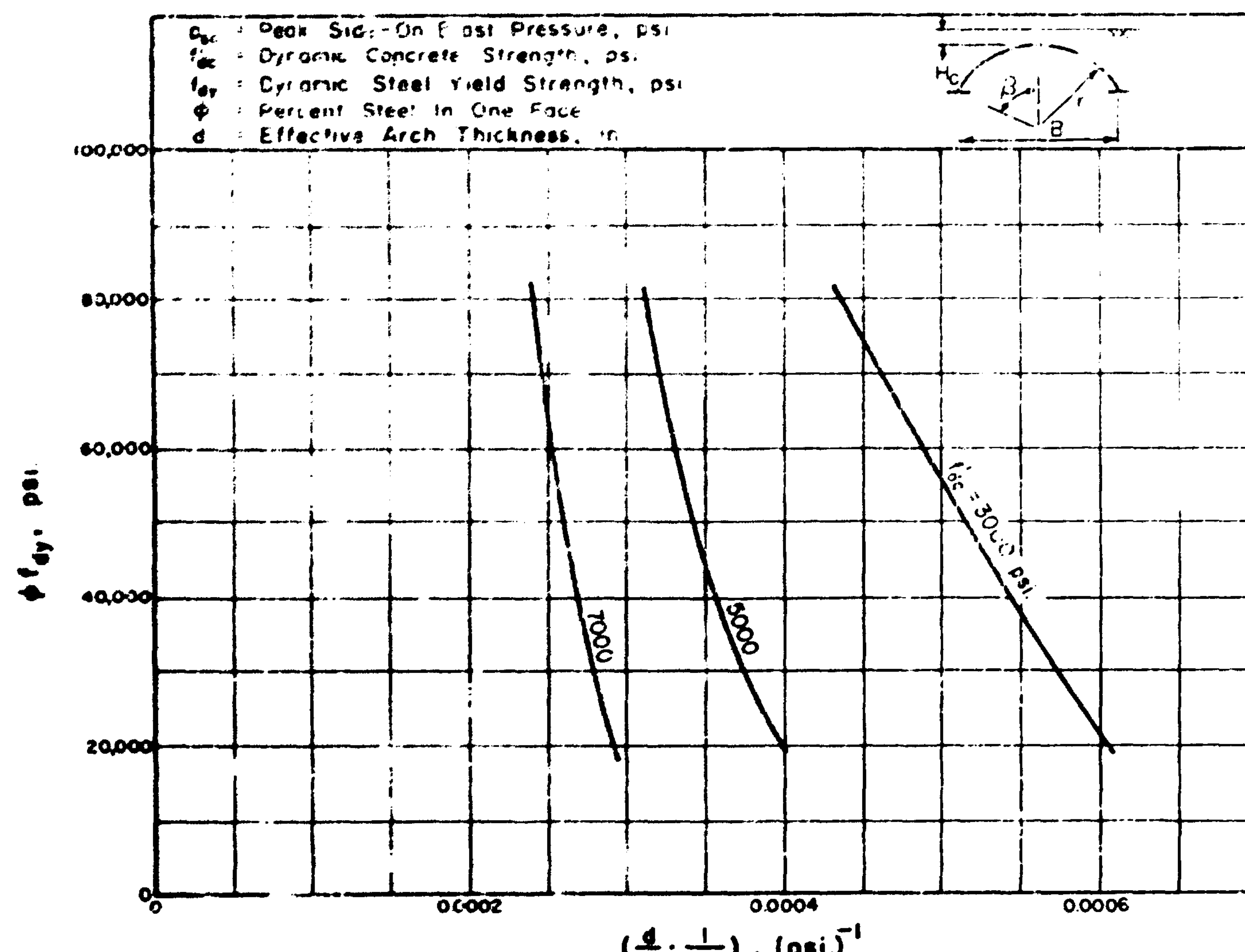


FIG. 5A-7.4 REQUIRED THICKNESS OF PARTIALLY-BURIED (H_c=0 → 0.062 B) REINFORCED CONCRETE ARCHES (μ = 1.3)

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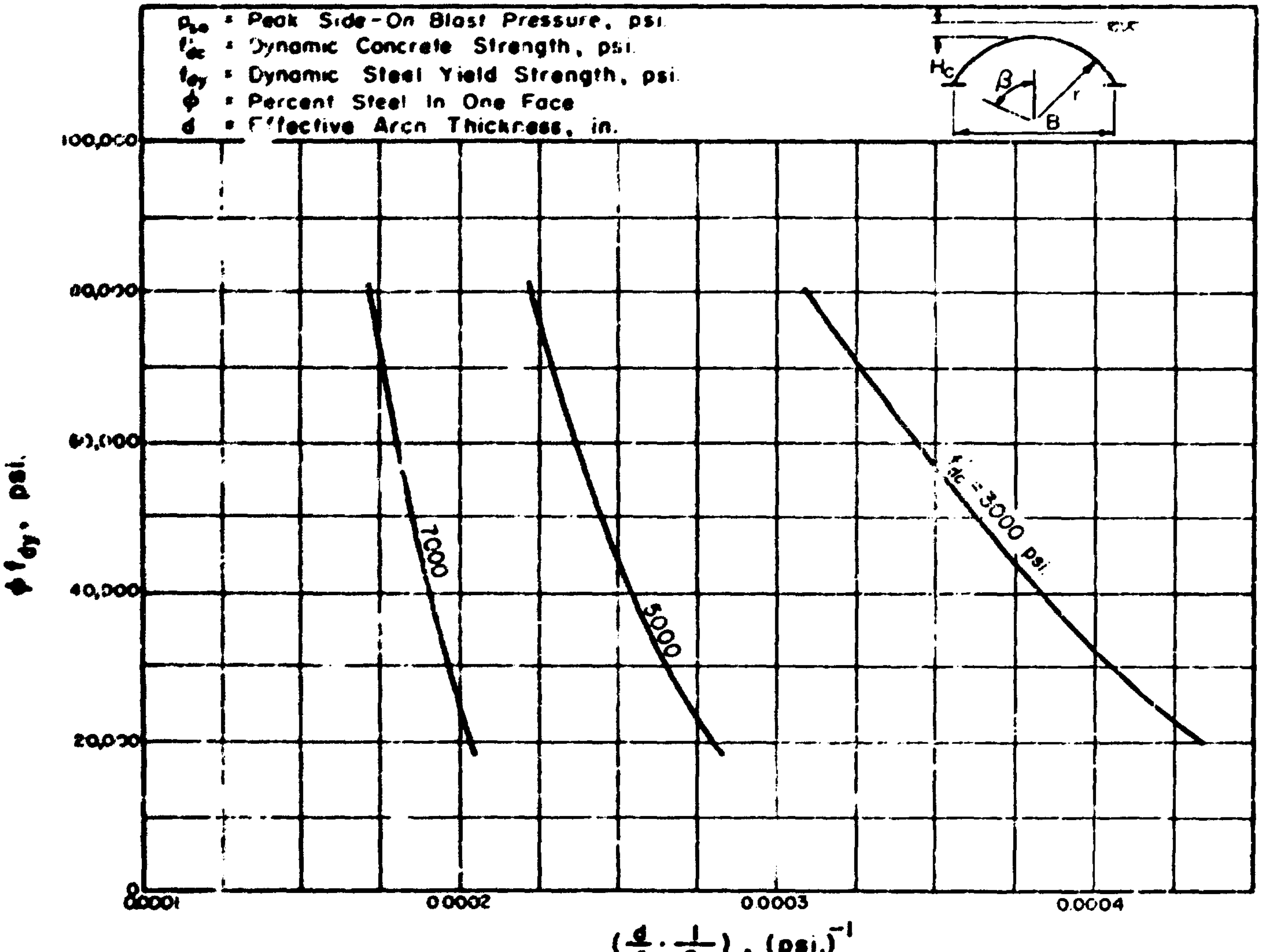
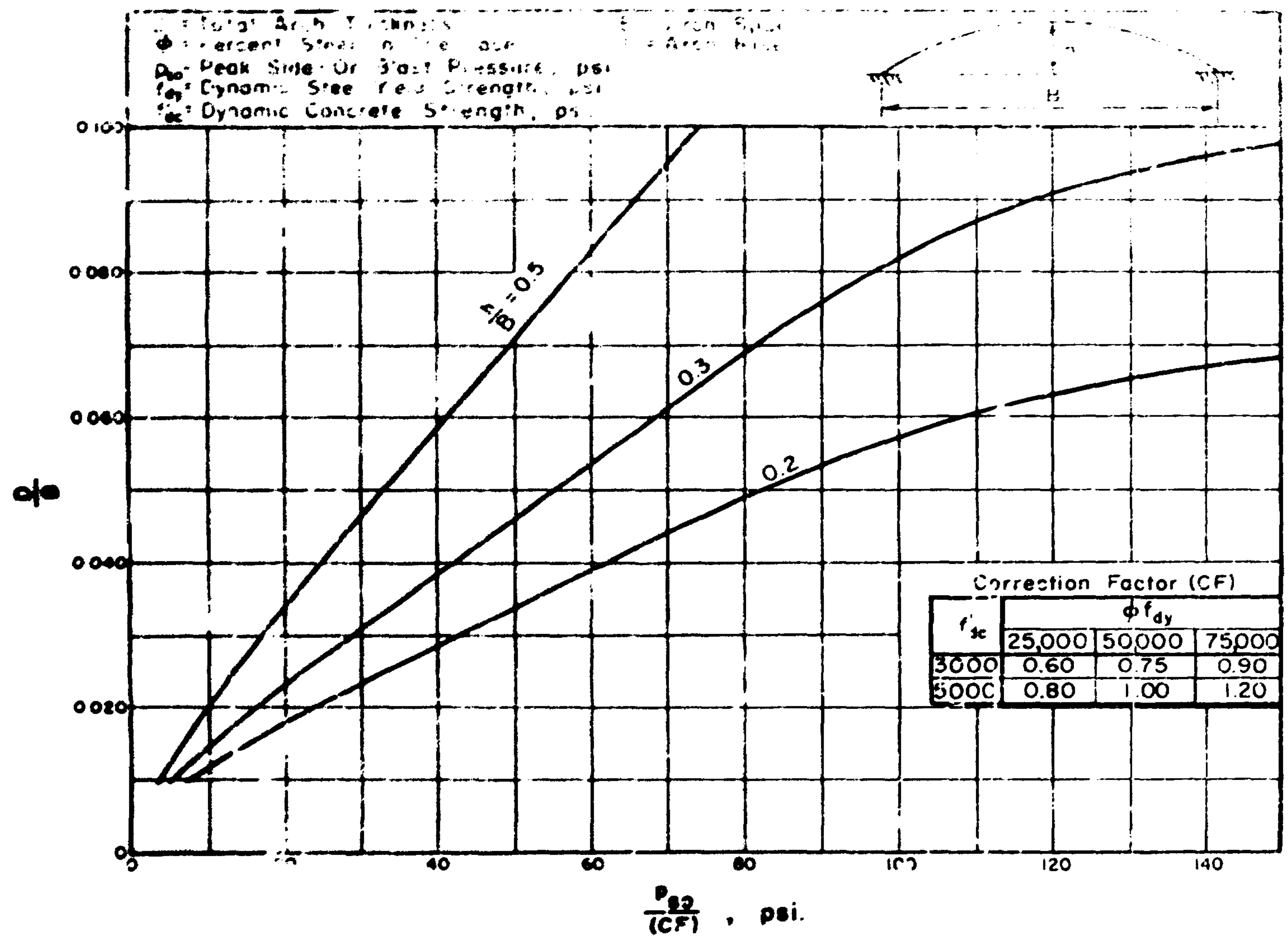


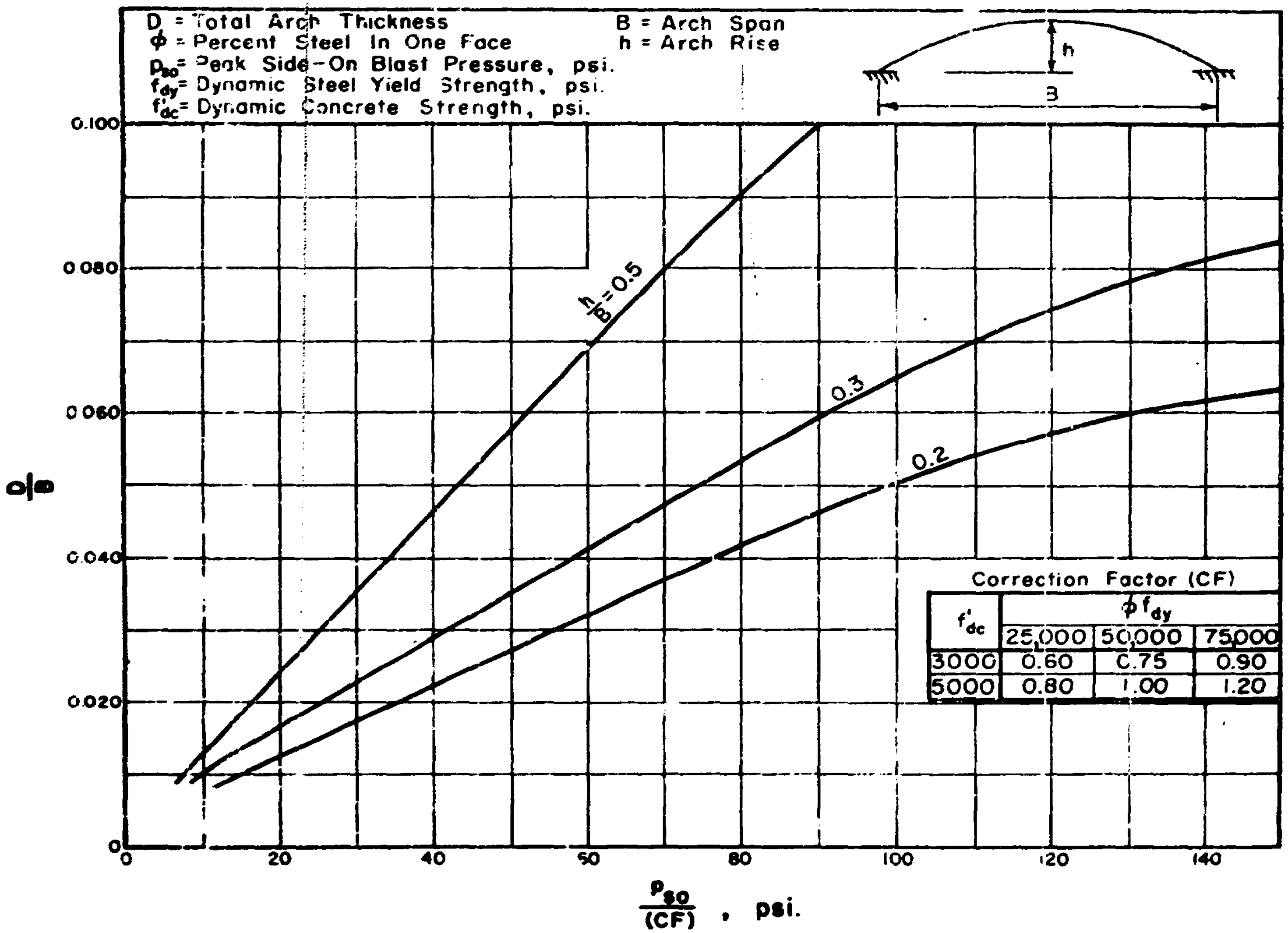
FIG. 5A-7.5 REQUIRED THICKNESS OF PARTIALLY-BURIED (H_c=0 → 0.062 B) REINFORCED CONCRETE ARCHES (μ = 3.0)



Correction Factor (CF)

f'_{dc}	ϕf_{dy} 25,000	50,000	75,000
3000	0.60	0.75	0.90
5000	0.80	1.00	1.20

FIG. 5A-7.6 REQUIRED THICKNESS OF ABOVEGROUND REINFORCED CONCRETE ARCHES ($\mu = 1.3$)



Correction Factor (CF)

f'_{dc}	ϕf_{dy} 25,000	50,000	75,000
3000	0.60	0.75	0.90
5000	0.80	1.00	1.20

FIG. 5A-7.7 REQUIRED THICKNESS OF ABOVEGROUND REINFORCED CONCRETE ARCHES ($\mu = 3.0$)

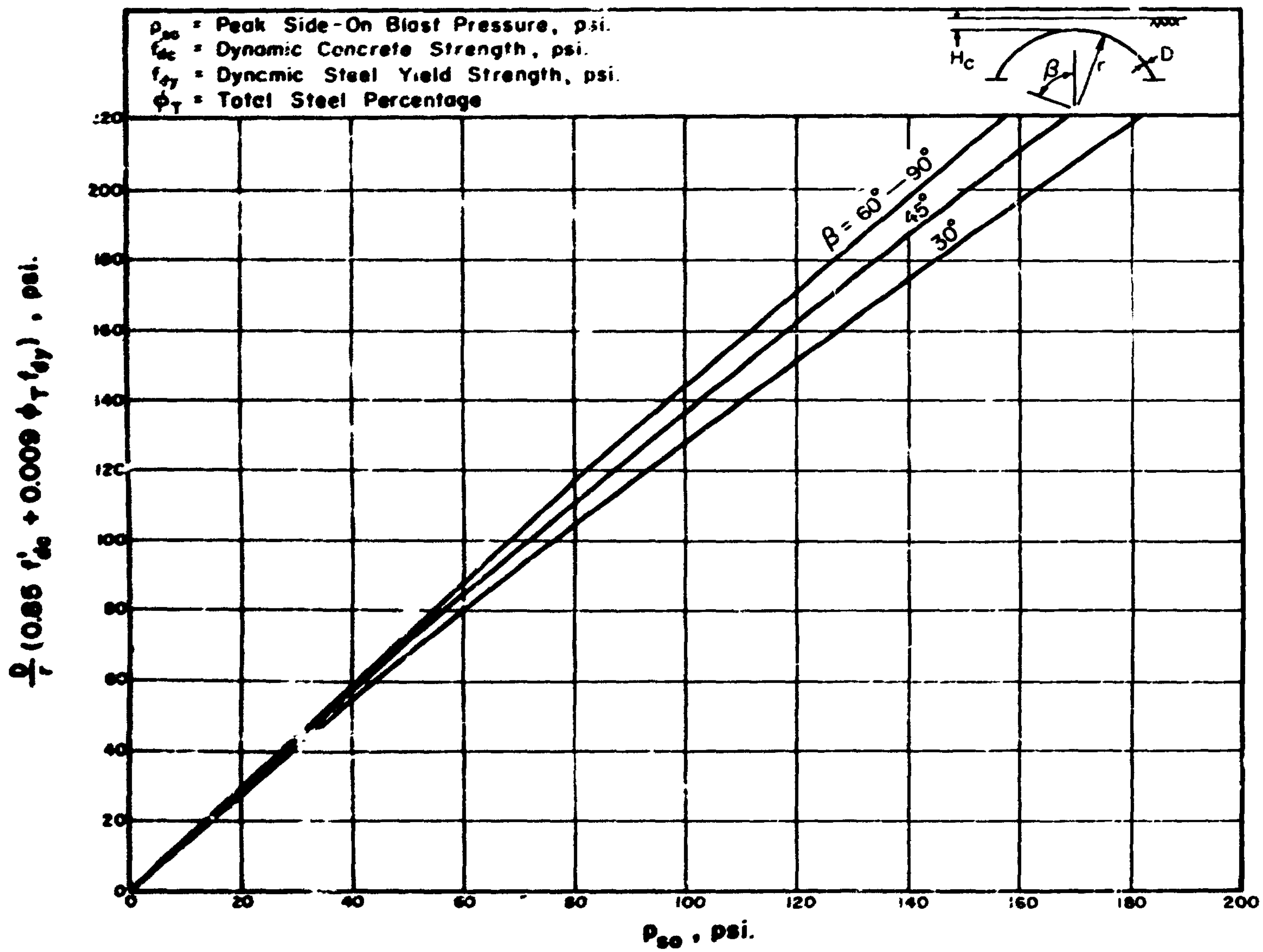


FIG. 5A-8.1 REQUIRED THICKNESS OF PARTIALLY-BURIED R/C DOMES (μ = 1.3)

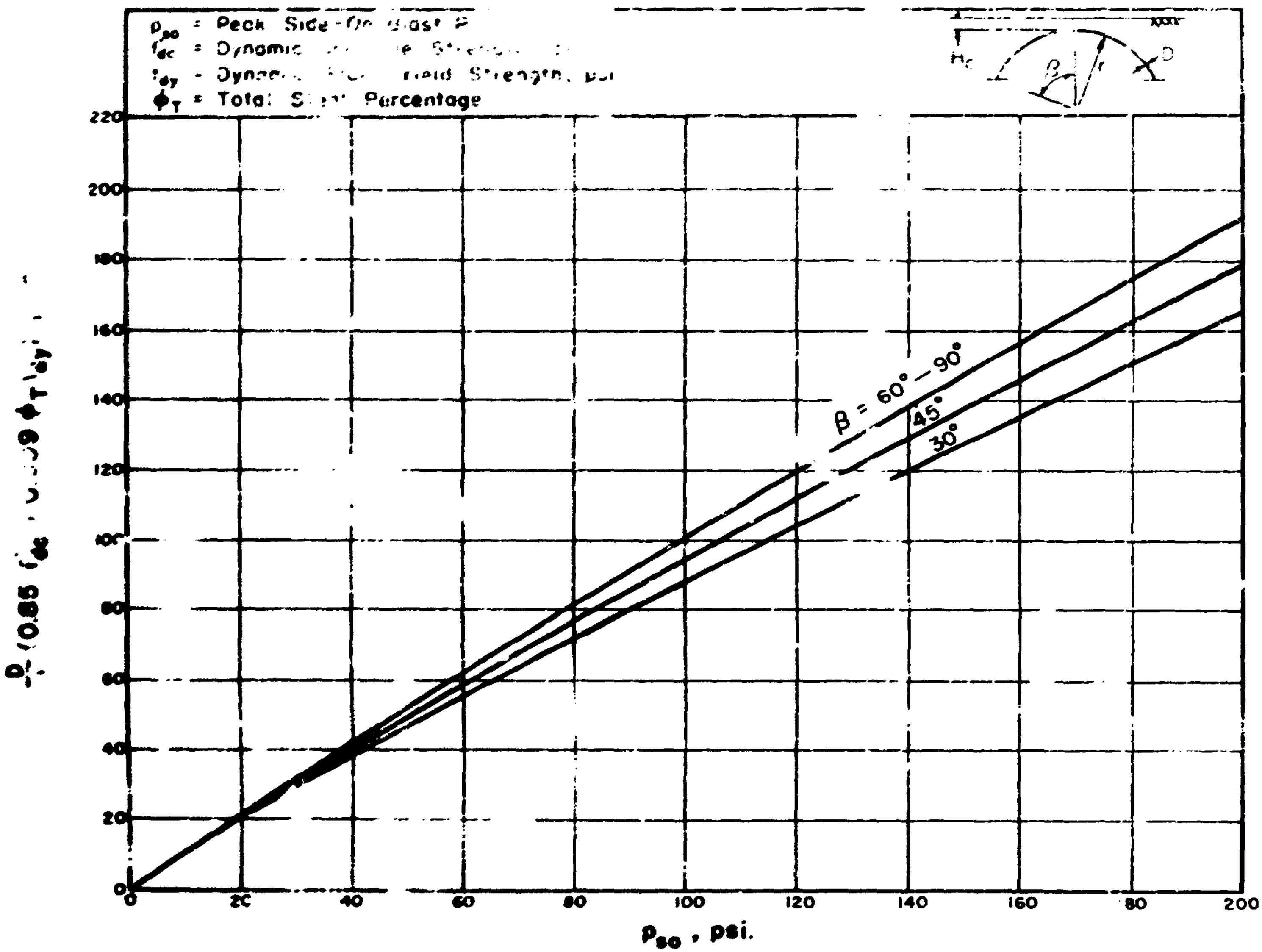


FIG. 5A-8.2 REQUIRED THICKNESS OF PARTIALLY-BURIED R/C DOMES (μ=3.0)

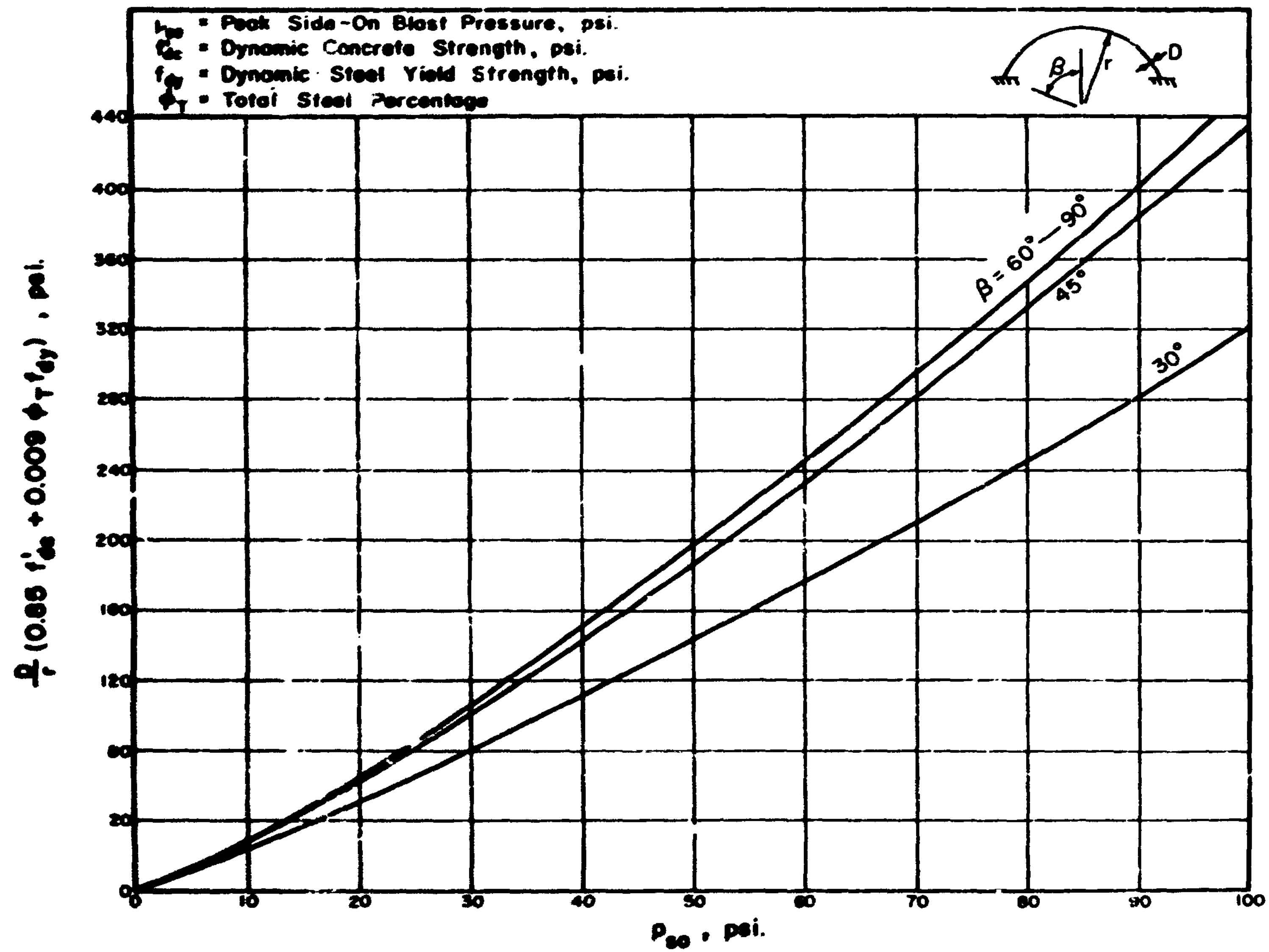


FIG. 5A-8.3 REQUIRED THICKNESS OF ABOVEGROUND R/C DOMES ($\mu = 1.3$)

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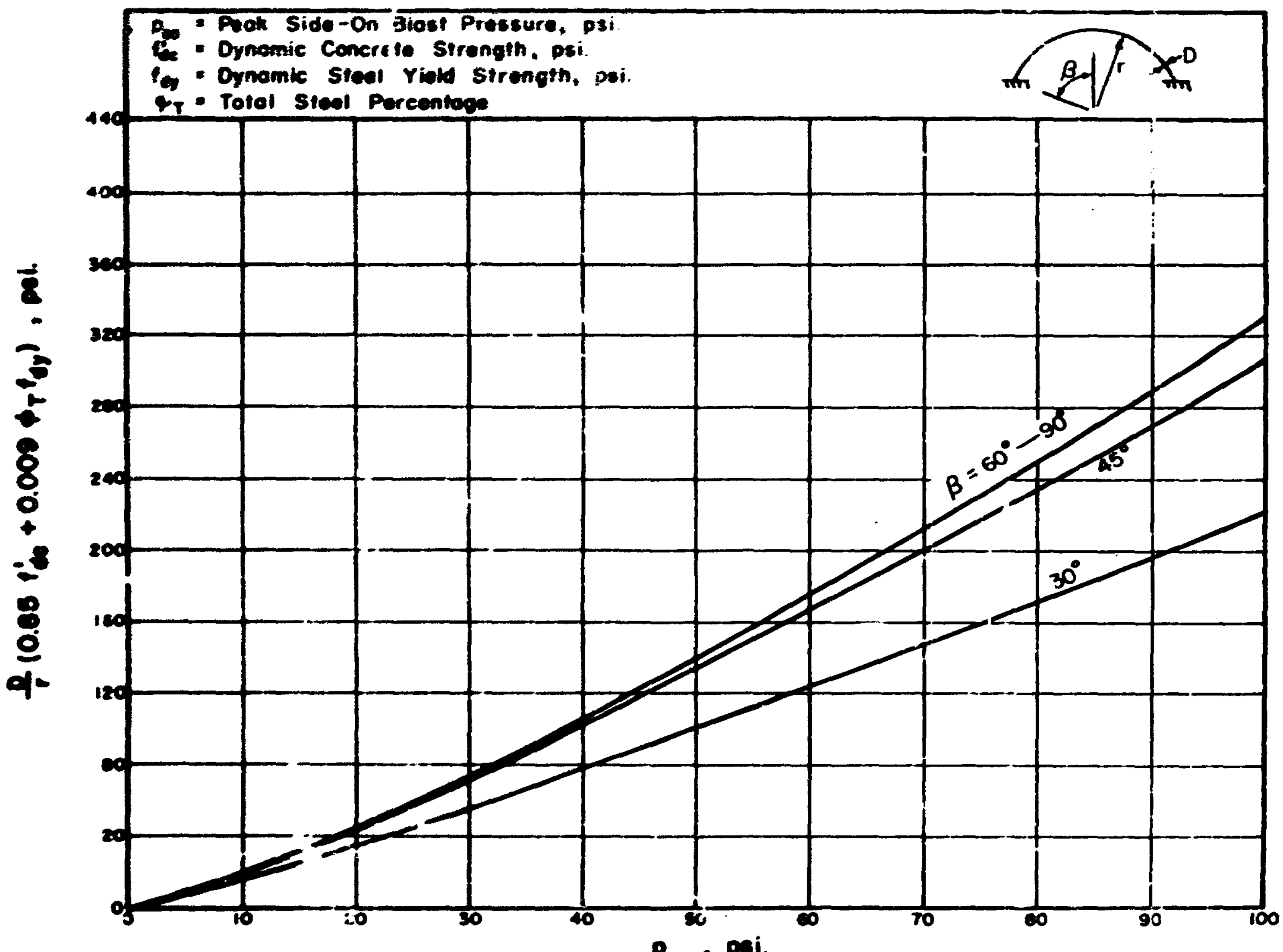


FIG. 5A-8.4 REQUIRED THICKNESS OF ABOVEGROUND R/C DOMES (μ=3.0)

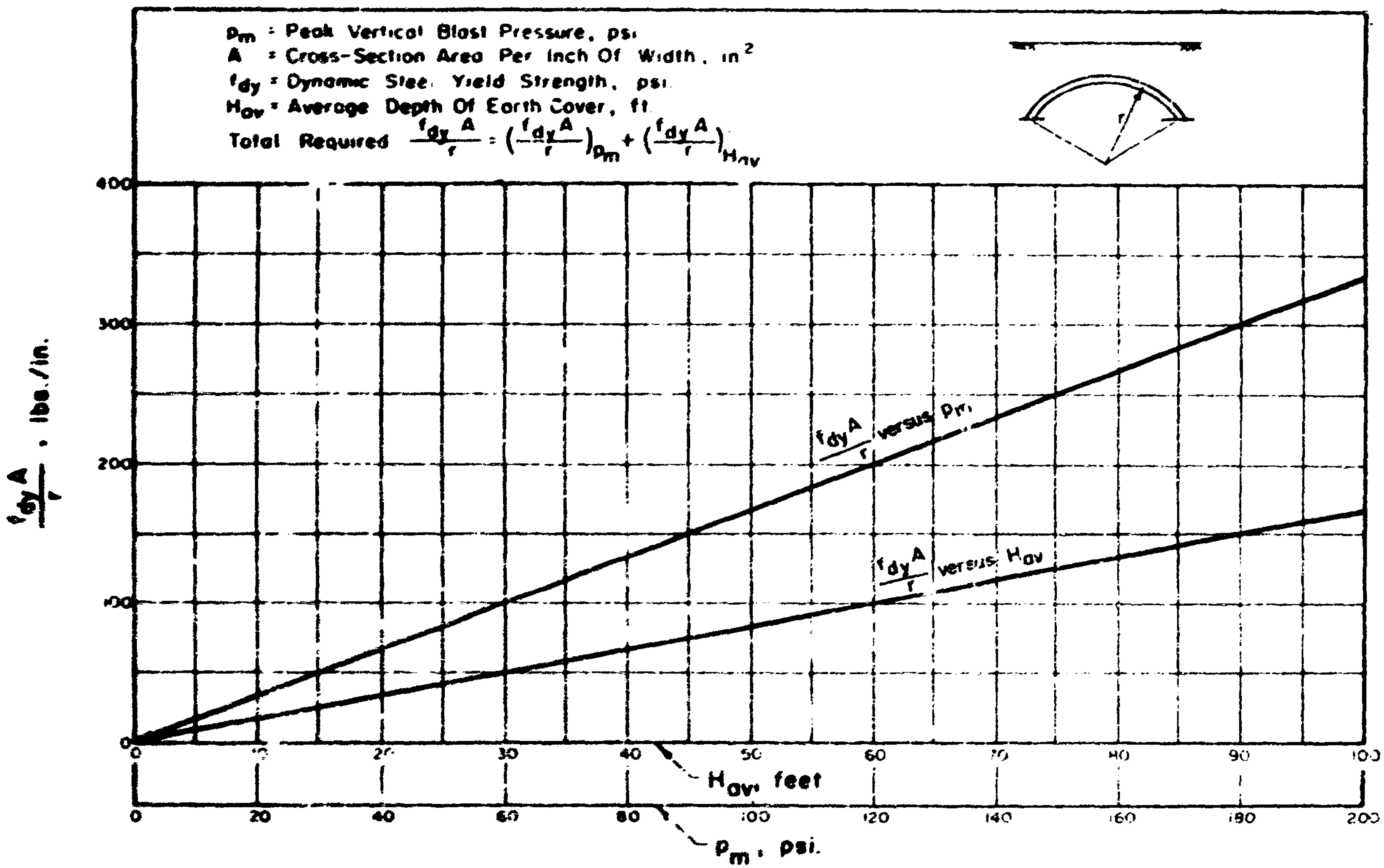


FIG. 5A-9.' REQUIRED AREA OF FULLY-BURIED STEEL ARCHES (μ = 1.3)

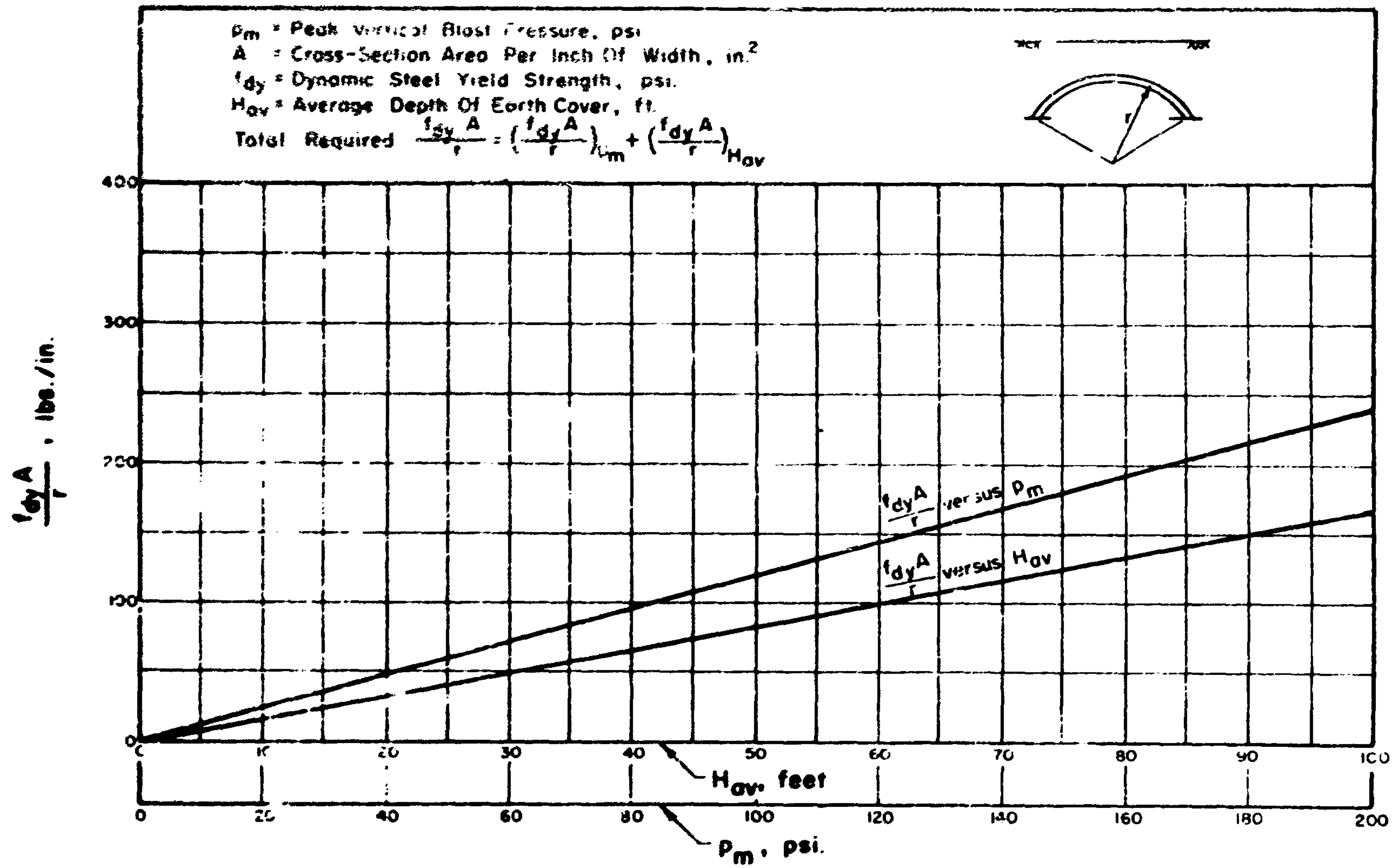


FIG. 5A-S.2 REQUIRED AREA OF FULLY-BURIED STEEL ARCHES (μ = 3.0)

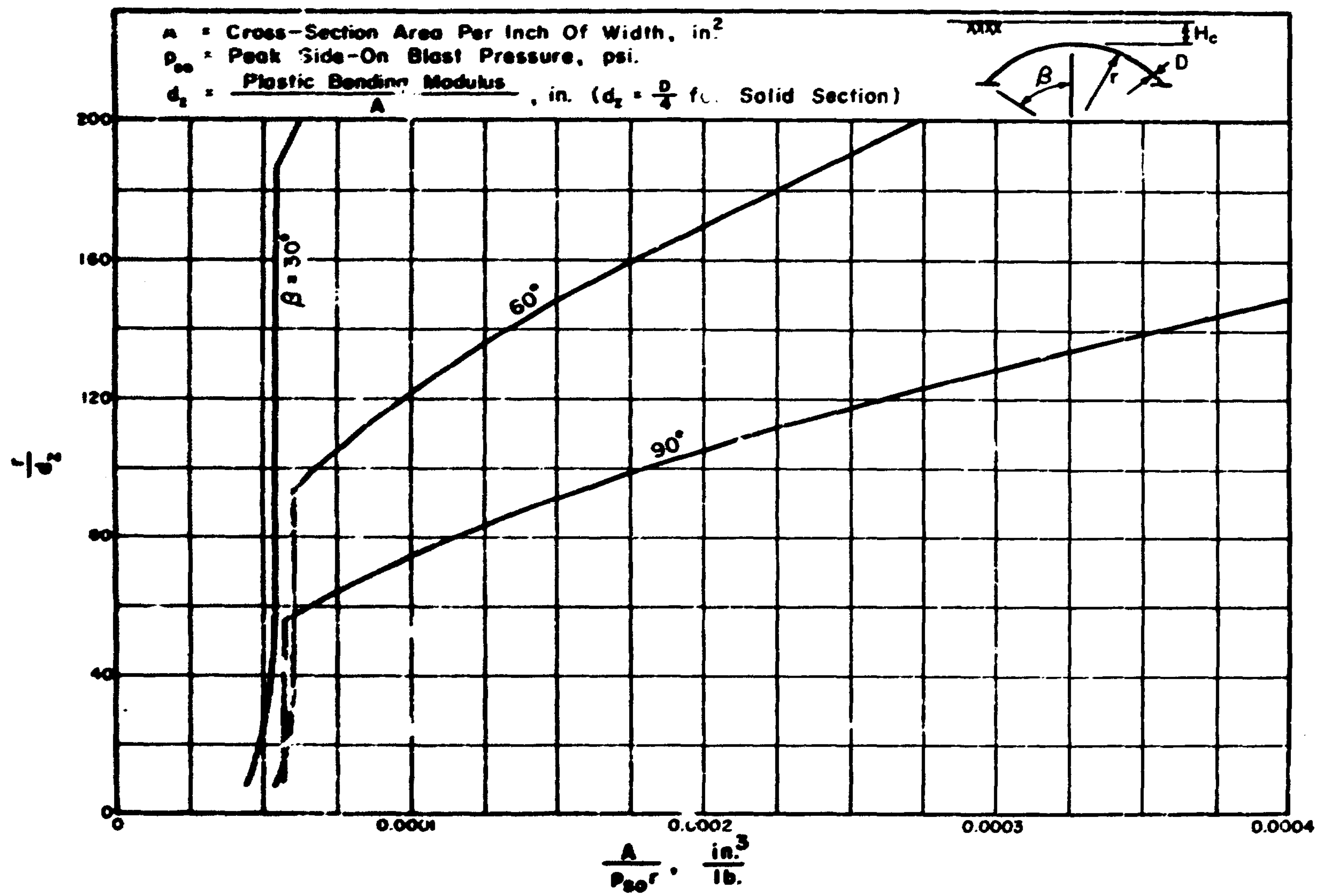


FIG. 5A-93 REQUIRED AREA OF PARTIALLY-BURIED STEEL ARCHES ($H_c = 0 \longrightarrow 0.062B$), ($\mu = 1.3$)

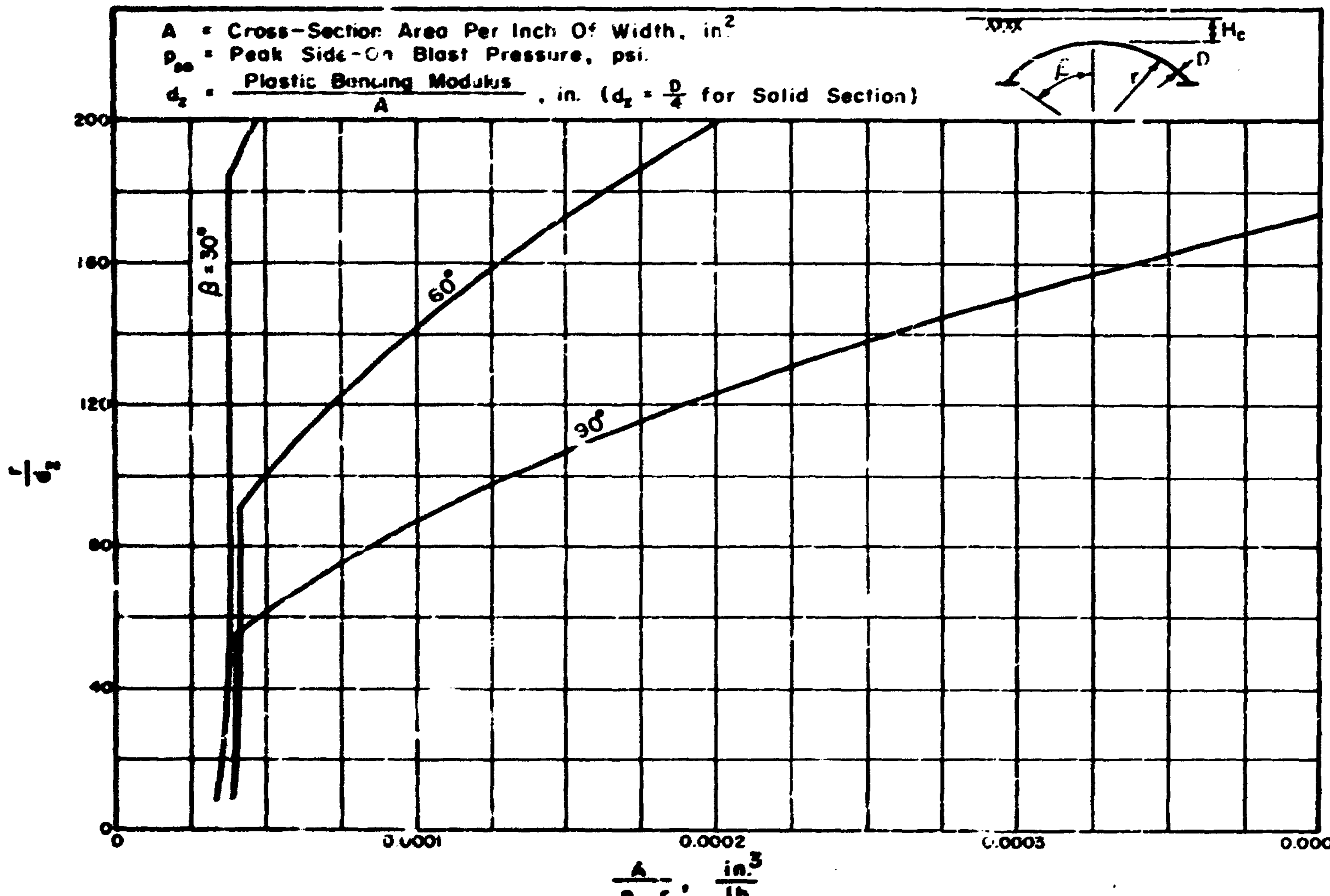


FIG. 5A-94 REQUIRED AREA OF PARTIALLY-BURIED STEEL ARCHES (H_c = 0 ⟶ 0.062 B), (μ = 3.0)

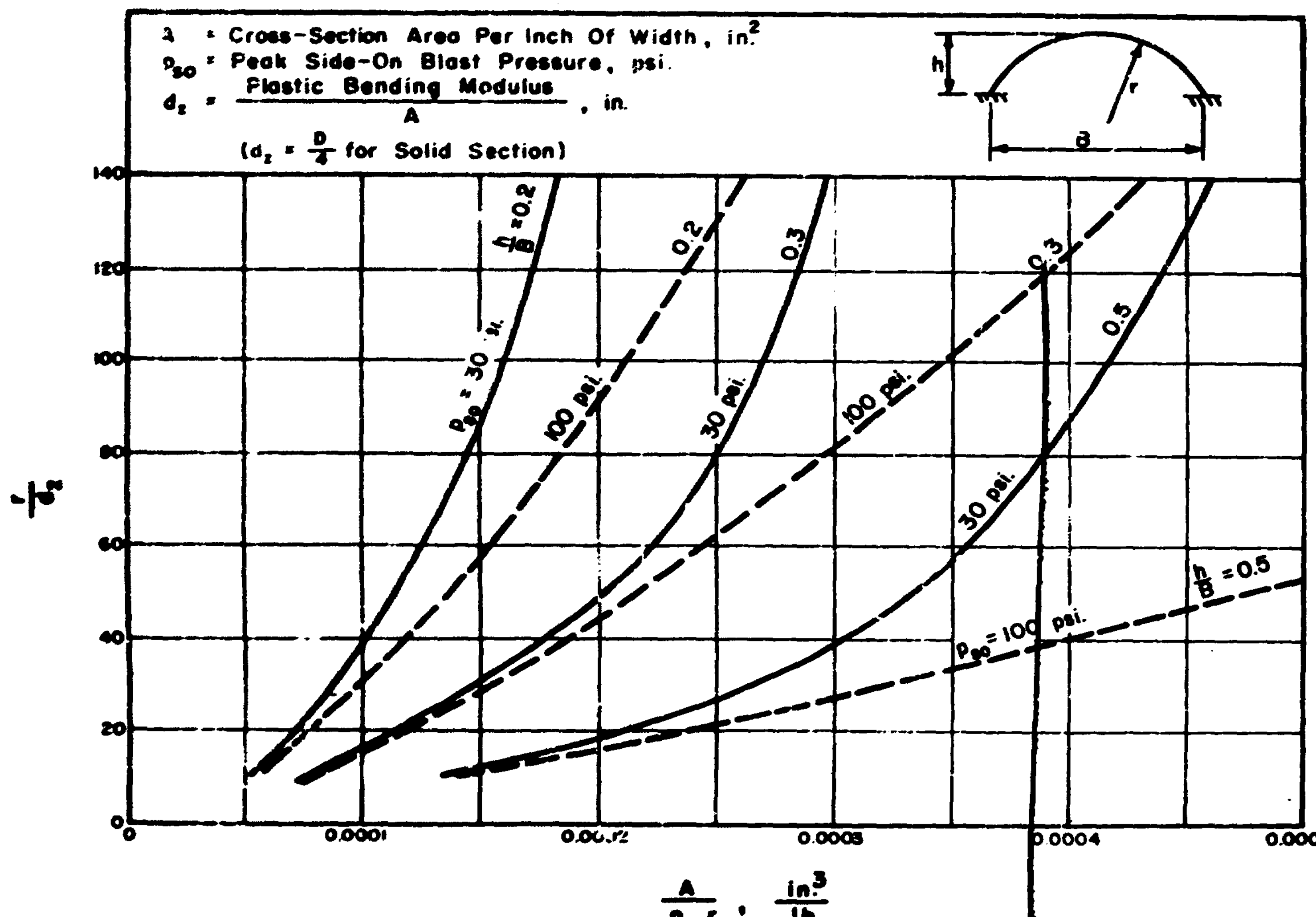


FIG. 5A-9.5 REQUIRED AREA OF ABOVEGROUND STEEL ARCHES
(μ = 1.3)

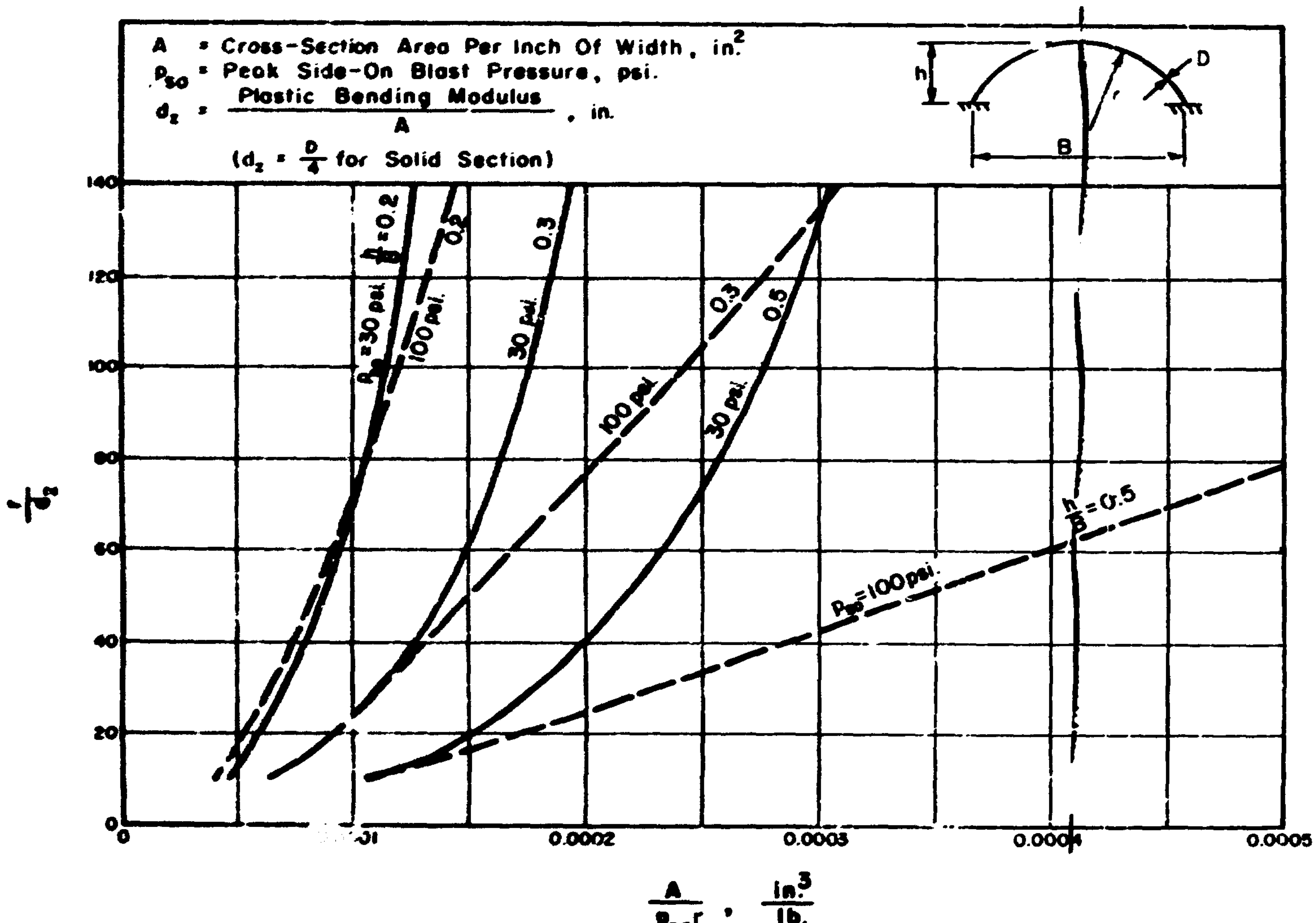


FIG. 5A-9.6 REQUIRED AREA OF ABOVEGROUND STEEL ARCHES
(μ = 3.0)

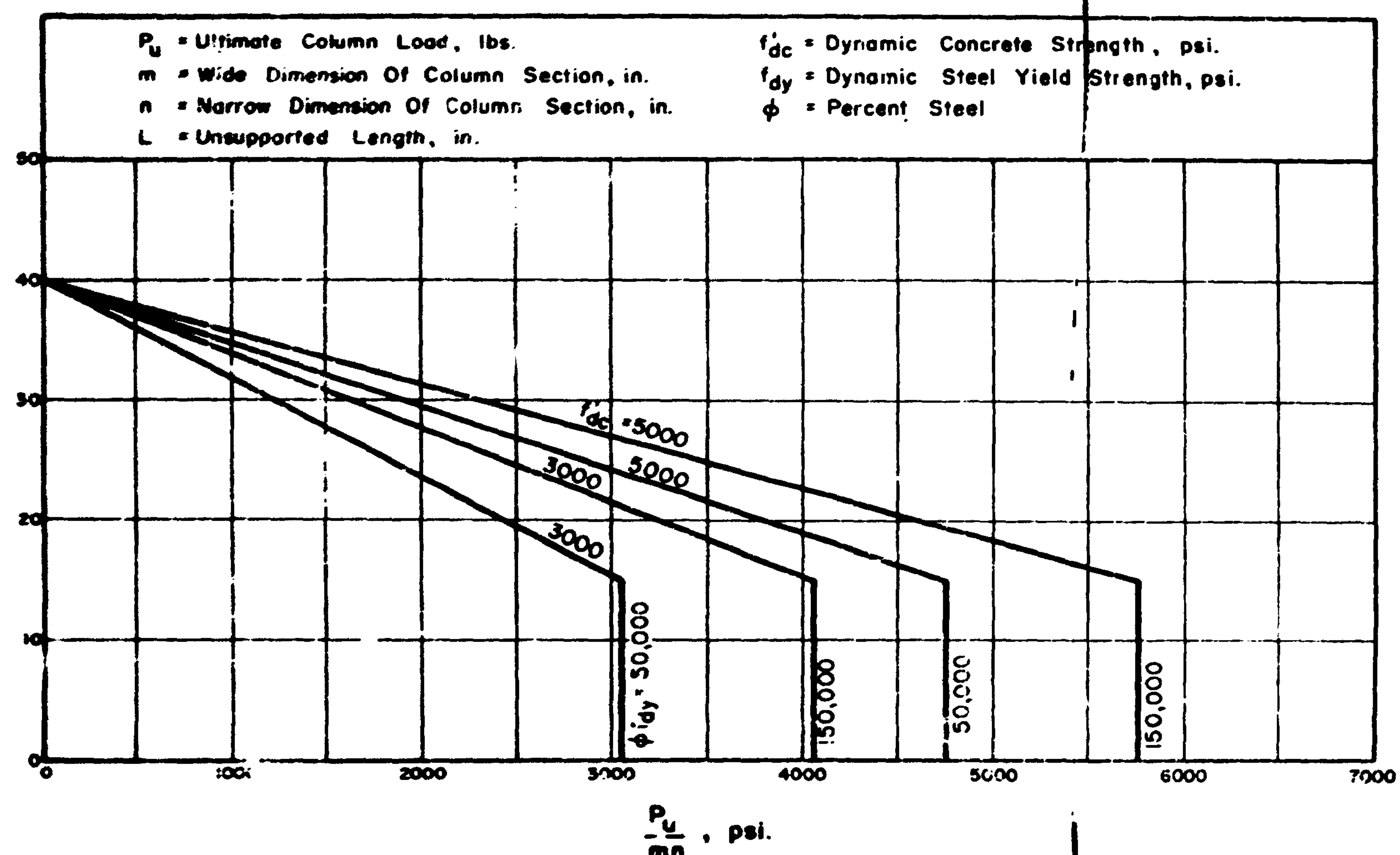


FIG. 5A-10.1 STRENGTH OF AXIALLY-LOADED RECTANGULAR REINFORCED CONCRETE COLUMNS

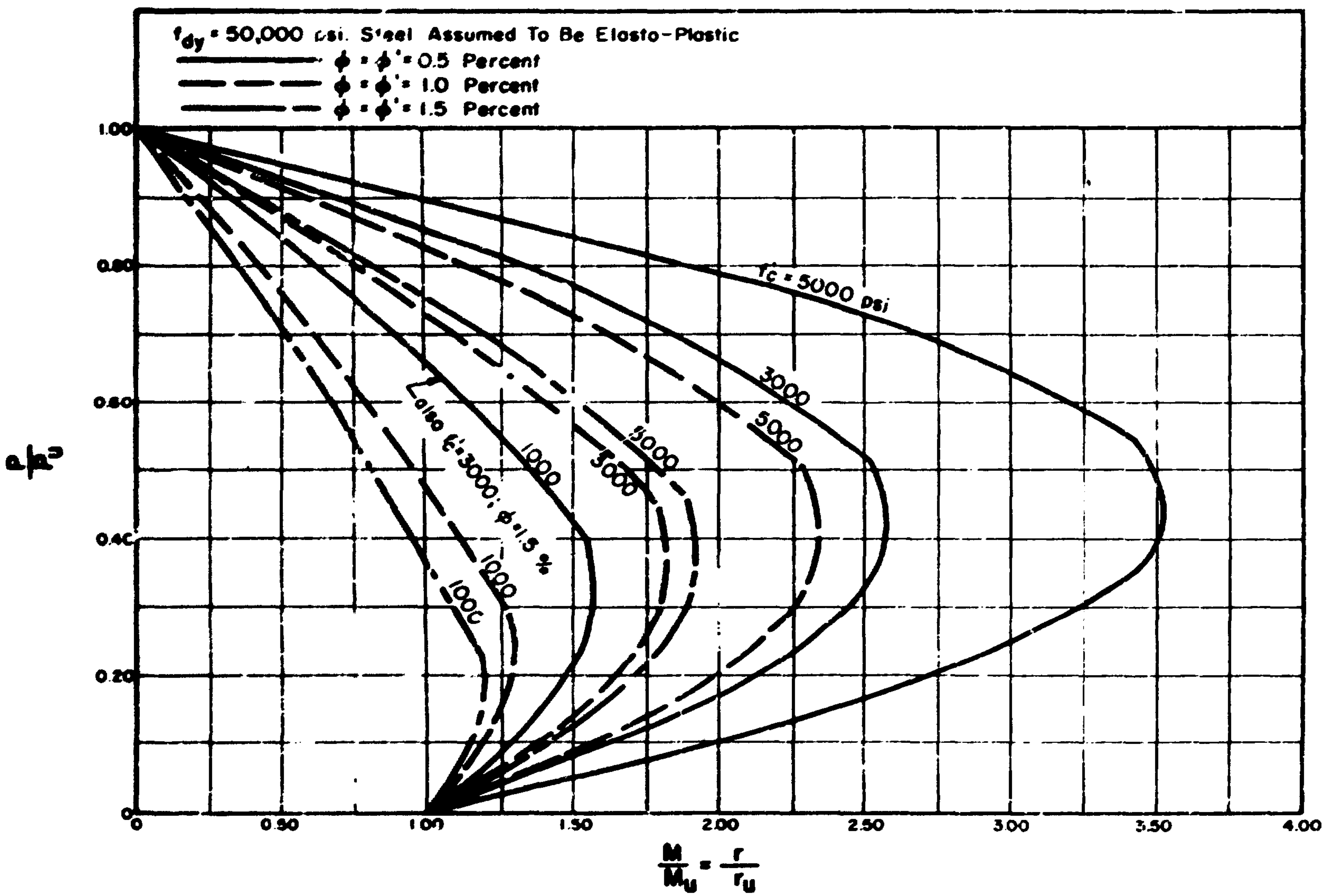


FIG. 5A-10.2 STRENGTH OF REINFORCED CONCRETE BEAM-COLUMNS

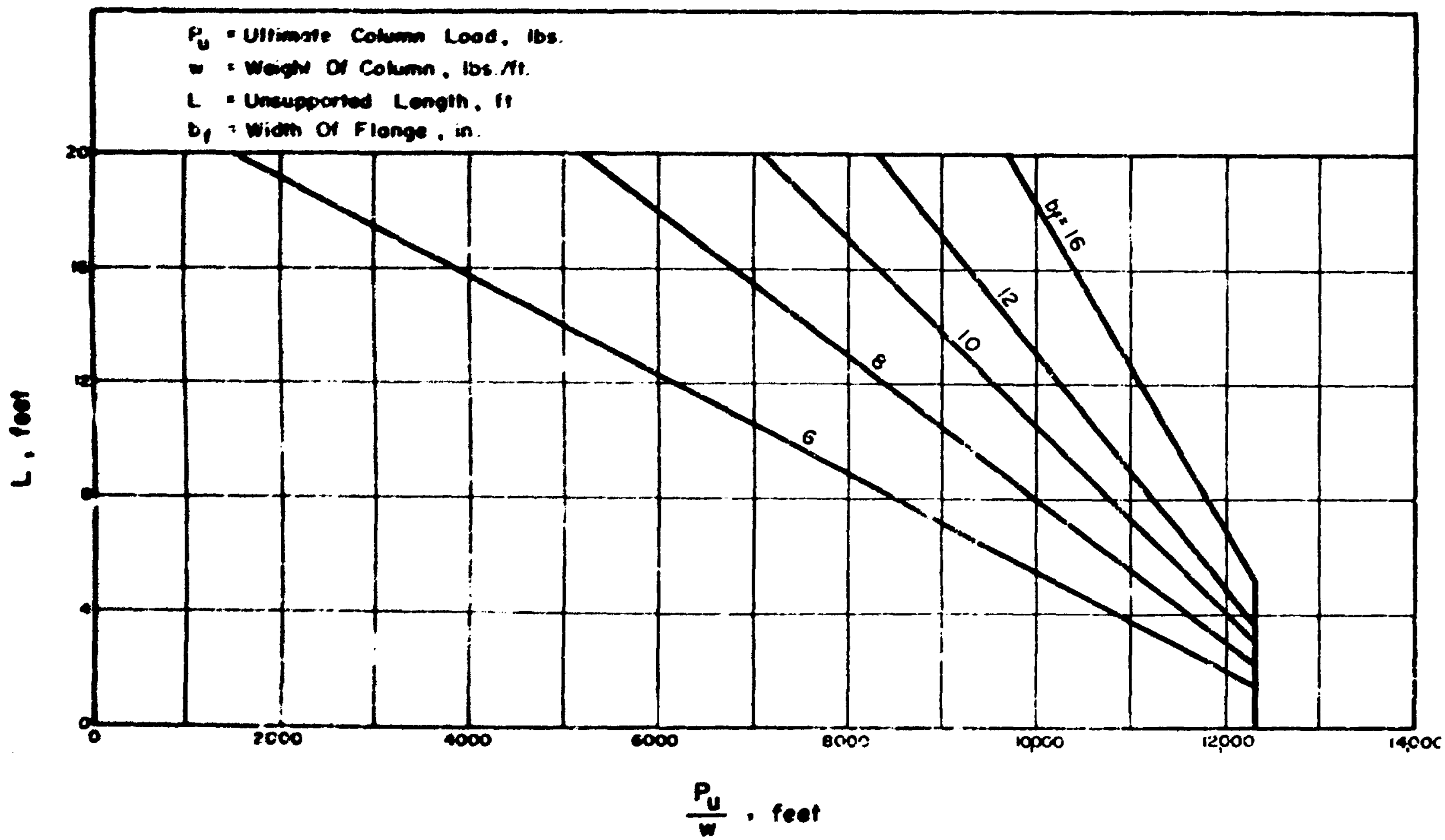


FIG. 5A-10.3 ULTIMATE STRENGTH OF AXIALLY-LOADED I-SECTION STEEL COLUMNS

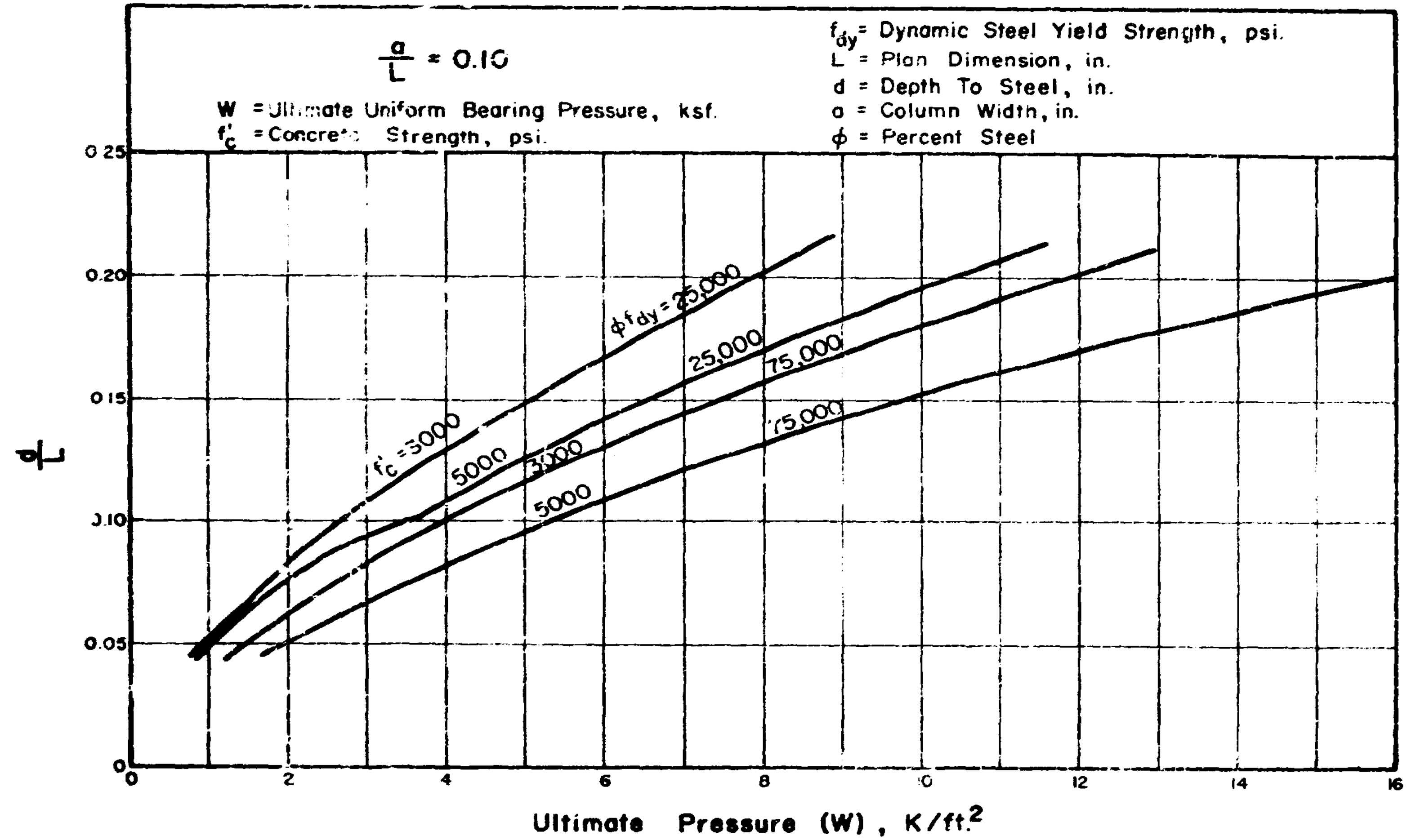


FIG. 5A-11.1 RESISTANCE OF SQUARE COLUMN FOOTINGS

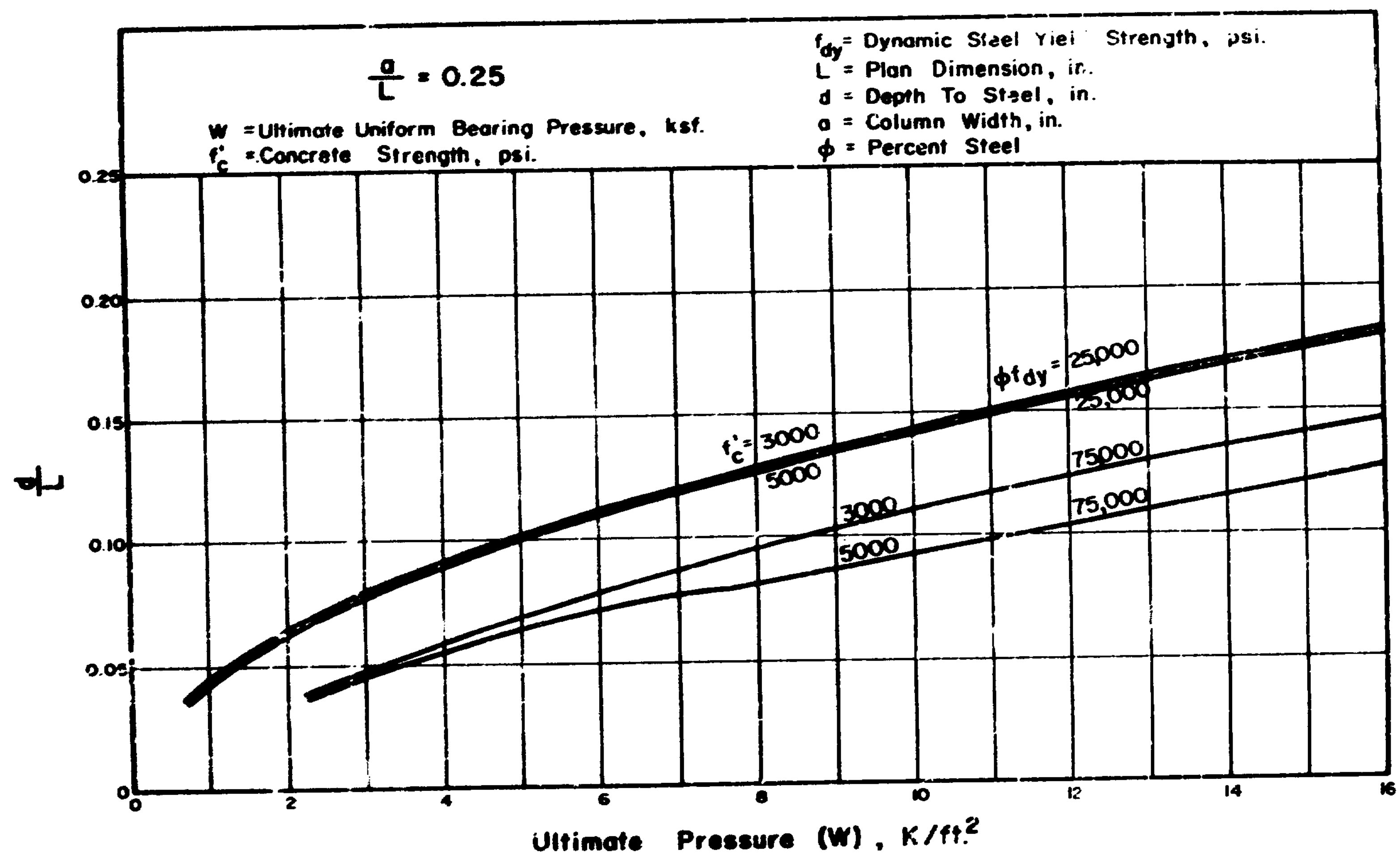


FIG. 5A-11.2 RESISTANCE OF SQUARE COLUMN FOOTINGS

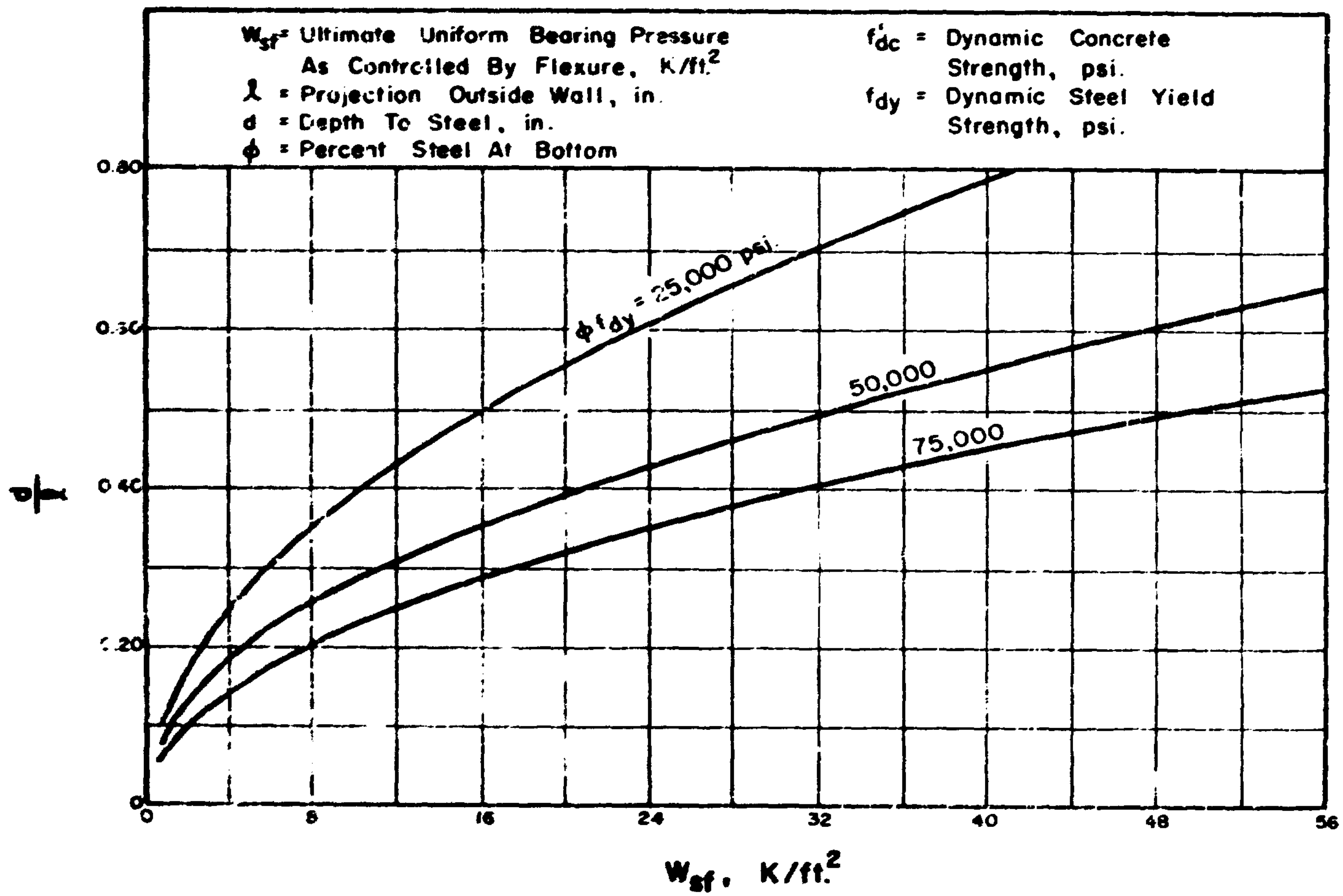


FIG. 5A-11.3 FLEXURAL RESISTANCE OF WALL FOOTINGS

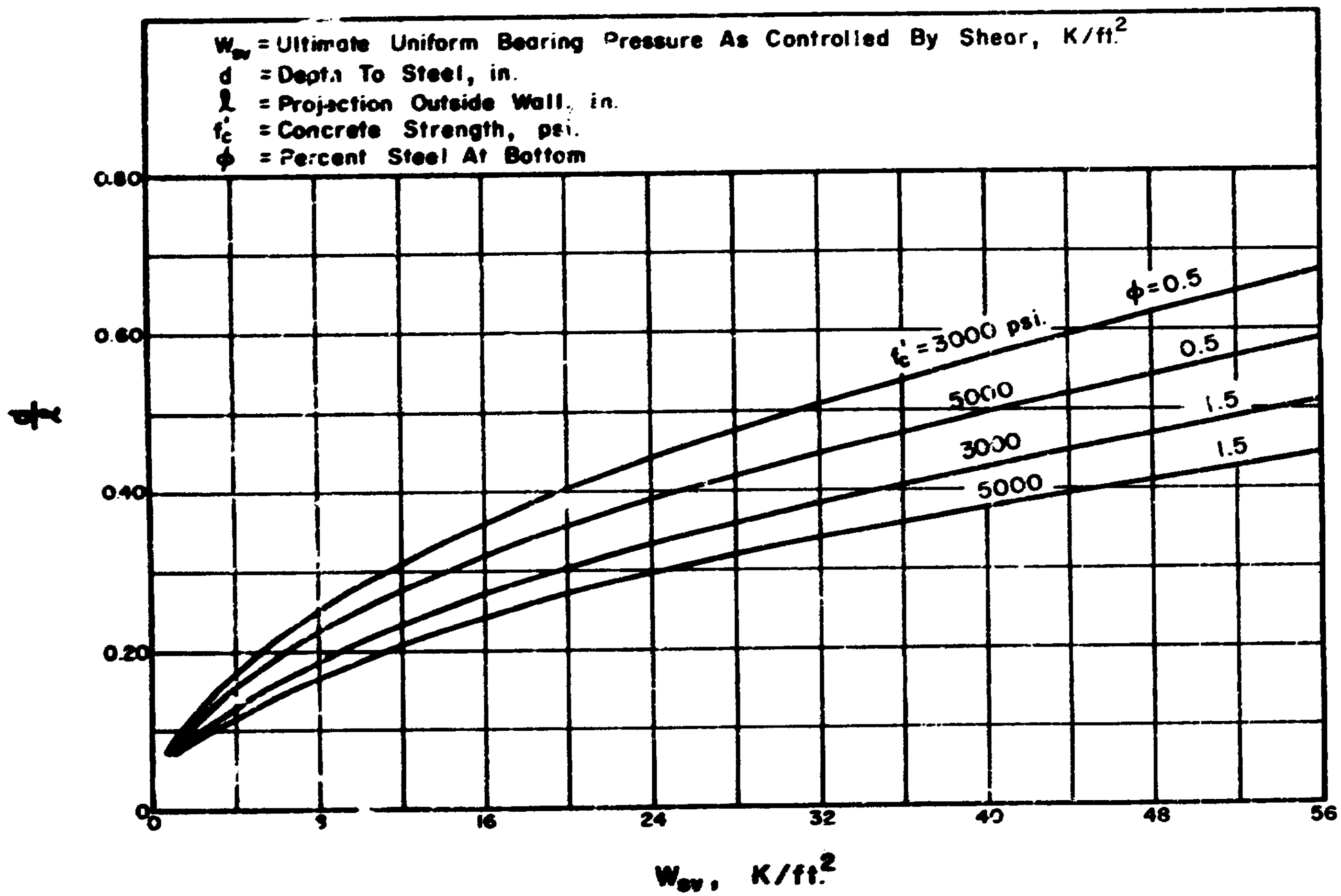


FIG. 5.4-11.4 SHEAR RESISTANCE OF WALL FOOTINGS WITHOUT WEB REINFORCEMENT.

APPENDIX 5B

DYNAMIC PROPERTIES OF MATERIALS AND STRUCTURAL ELEMENTS

APPENDIX 5B. DYNAMIC PROPERTIES OF MATERIALS AND STRUCTURAL ELEMENTS

5B.1 STEEL

The stresses given in this and the following sections a[illegible] those recommended for dynamic design. They include appropriate increas[illegible] for the effect of rate of straining and should b[illegible] used to compute the ultimate dynamic strength of structural elements.

Structural Carbon Steel (ASTM A7)

f_{dy} = dynamic yield point in tension or compression
= 42,000 psi

v_{dy} = dynamic yield point in shear = 25,000 psi

Welds:

f_{dy} = 42,000 psi
v_{dy} = 29,000 psi

Rivets (ASTM A141):

f_{dy} = 40,000 psi
v_{dy} = 30,000 psi
f_{by} = 60,000 psi (bearing - single shear)
80,000 psi (double shear)

Bolts (ASTM A307):

f_{dy} = 32,000 psi
v_{dy} = 19,000 psi
f_{by} = 40,000 psi

Reinforcing Steel

Intermediate grade, f_{dy} = 52,000 psi
Structural grade, f_{dy} = 44,000 psi

Other Steels

High Strength Rivets (ASTM A195):

f_{dy} = 60,000 psi
v_{dy} = 40,000 psi
f_{by} = 80,000 psi

High Strength Bolts (ASTM A325):

f_{dy} = 50,000 psi

v_{dy} = 30,000 psi

f_{by} = 60,000 psi

High Strength Alloys:

It is recommended that the dynamic yield strength, f_{dy}, be taken as 1.25 times the minimum specified static yield strength but no more than 90% of the ultimate strength. The shear yield strength may be taken as 60% of the tensile yield strength.

5B.2 STEEL BEAMS

<u>Flexural Strength of Beam Sections</u>:

$$M_p = f_{dy} Z \approx 1.1 f_{dy} S \text{ (for I-sections)}$$

where Z = plastic modulus

S = section modulus

M_p = ultimate moment capacity

<u>Shear Strength of Beam Sections</u>:

$$V_p = v_{dy} A_w$$

where A_w = area of web

V_p = ultimate shear capacity

<u>Buckling of Beams</u>:

If plastic deformation is to occur ($\mu > 1$) plastic buckling must be prevented and the following requirements must be satisfied:

<u>Minimum Thicknesses</u>:

$\frac{b}{t_f} < 17$

$\frac{a}{t_w} < 70$ (without longitudinal stiffeners)

$\frac{b_s}{t_s} < 8\ 1/2$

where b = flange width

t_f = flange thickness

a = web depth

t_w = web thickness

b_s = stiffener width

t_s = stiffener thickness

Lateral support:

$$\ell_{cr} < (60-40\frac{M}{M_p})\, r_y$$

where, ℓ_{cr} = unbraced length on one side of plastic hinge

M = moment at end of ℓ_{cr} away from hinge

r_y = lateral radius of gyration of compression flange

Resistance:

Uniform Load:	Flexure	Shear
Simply supported or hinged,	$R_f = 8M_p/L$	$R_s = 2V_p$
Fixed one end and hinged one end,	$R_f = \frac{8M_p}{L} + \frac{4M_p^n}{L}$	$R_s = 2V_p - \frac{2M_p^n}{L}$
Fixed both ends,	$R_f = \frac{8M_p}{L} + \frac{8M_p^n}{L}$	$R_s = 2V_p$
Concentrated Load at midspan:		
Simply supported,	$R_f = 4M_p/L$	$R_s = 2V_p$
Fixed one end and hinged one end,	$R_f = \frac{4M_p}{L} + \frac{2M_p^n}{L}$	$R_s = 2V_p - \frac{2M_p^n}{L}$
Fixed both ends,	$R_f = \frac{4M_p}{L} + \frac{4M_p^n}{L}$	$R_s = 2V_p$

Where, R = total resistance (lb. or kips)

L = span

M_p = ultimate bending strength at midspan

M_p^n = ultimate negative bending strength at support

V_p = ultimate shear strength

Stiffness:

	Uniform Load	Concentrated Load at Mid-span
Simply supported,	$k = \frac{384EI}{5L^3}$	$k = \frac{48EI}{L^3}$

Fixed one end and	$k = \frac{160EI}{L^3}$ *	$k = \frac{106EI}{L^3}$ *
Fixed both ends,	$k = \frac{307EI}{L^3}$ *	$k = \frac{192EI}{L^3}$

Where E = modulus of elasticity

I = moment of inertia

Note: Yield deflection = $x_e = R/k$

*These values are "effective" stiffnesses which convert the tri-linear resistance function into an equivalent bi-linear function with the same energy absorption capacity at yield.

<u>Natural Period</u> (Seconds):

Uniformly distributed mass:

Simply supported, $T = 0.64L^2 \sqrt{\frac{w}{EIg}}$

Fixed one end and hinged one end, $T = 0.42L^2 \sqrt{\frac{w}{EIg}}$

Fixed both ends, $T = 0.28L^2 \sqrt{\frac{w}{EIg}}$

where w = supported weight (including beam) per unit length

Concentrated mass at midspan:

Simply supported, $T = 0.91 \sqrt{\frac{W_c L^3}{EIg}}$

Fixed one end and hinged one end, $T = 0.61 \sqrt{\frac{W_c L^3}{EIg}}$

Fixed both ends, $T = 0.45 \sqrt{\frac{W_c L^3}{EIg}}$

where W_c = total weight concentrated at midspan

5B.3 STEEL COLUMNS

<u>Axially-Loaded Columns</u>:

Strength:

$$f_{cr} = 45{,}000 - 200\,\frac{\alpha L}{r}\text{ , not to exceed } f_{dy}$$

where, f_{cr} = maximum allowable stress intensity

$\frac{L}{r}$ = slenderness ratio

α = classical length reduction factor (α = 1 for pinned ends)

Note: No ductility should be anticipated in columns unless $\alpha L/r < 15$.

Minimum Thicknesses: $\frac{b}{t_f} < 17 \qquad \frac{a}{t_w} < 43$

Stiffness: $k = \frac{AE}{L}$ where A = cross-section area

<u>Beam-Columns</u>:

Strength:

$$\frac{M_p'}{M_p} + \frac{P'}{f_{cr}A} = 1$$

where M_p' and P' are ultimate values of moment and thrust acting in combination

5B.4 CONCRETE

$f'_{dc} = 1.25\ f'_c$ where f'_c = standard 28-day compressive strength

f'_{dc} = dynamic compressive strength

$u_d = 0.15\ f'_c$ = allowable dynamic bond stress on deformed bars (A305)

$f_{dt} = 7.5\sqrt{f'_{dc}}$ = dynamic tensile strength

5B.5 REINFORCED CONCRETE BEAMS AND ONE-WAY SLABS

<u>Flexural Strength of Cross Sections</u>:

$$M_p = \frac{\varphi b d^2}{100} f_{dy}\left[1 - \frac{\varphi f_{dy}}{170 f'_{dc}}\right] \approx 0.009\ \varphi b d^2\ f_{dy}$$

where b = width of section

d = depth of section to steel

φ = steel percentage in tension face

<u>Shear Strength of Cross Section</u>:

Pure Shear:

$$V_p = 0.22\ bdf'_c\ ,\ \text{if unreinforced}$$

$$= (0.22\ bdf'_c)\lambda_s\ ,\ \text{if reinforced}$$

$$\lambda_s = \frac{1}{2}\left[1 + \frac{\varphi'_v}{10} f_{dy}/f'_c\right],\ \text{but}\ \lambda_s \not< 1.0$$

φ'_v = percent of steel (inclined at 45°) crossing a surface inclined at 45°.

Diagonal Tension:

The strength in diagonal tension depends upon the type of element and loading. See below.

<u>Flexural Resistance</u>:

Uniform Load:

Simply supported, $R_f = 8M_p/L$

$$r_f = 0.072\varphi f_{dy}\left(\frac{d}{L}\right)^2$$

Fixed one end and hinged one end, $R_f = \frac{8M_p}{L} + \frac{4M_p^n}{L}$

$$r_f = 0.072\left(\varphi + \frac{\varphi'}{2}\right) f_{dy}\left(\frac{d}{L}\right)^2$$

Fixed both ends, $R_f = \frac{8M_p}{L} + \frac{8M_p^n}{L}$

$$r_f = 0.072(\varphi + \varphi')\ f_{dy}\left(\frac{d}{L}\right)^2$$

Concentrated Load at Midspan:

Simply supported,

$$R_f = 4M_p/L$$

$$r_f = 0.036\varphi \frac{bd^2}{L} f_{dy}$$

Fixed one end and hinged one end:

$$R_f = \frac{4M_p}{L} + \frac{2M_p^n}{L}$$

$$r_f = 0.036(\varphi + \frac{\varphi'}{2}) \frac{bd^2}{L} f_{dy}$$

Fixed both ends,

$$R_f = \frac{4M_p}{L} + \frac{4M_p^n}{L}$$

$$r_f = 0.036(\varphi + \varphi') \frac{bd^2}{L} f_{dy}$$

Where, φ' = negative steel percentage at support
r_f = resistance per unit area of top surface = R_f/bL

Pure Shear Resistance:

Note: Critical section need be taken no closer to support than 0.5d, or 0.1L, whichever is smaller.

	Uniform Load $\frac{d}{L} \geq 0.2$	Uniform Load $\frac{d}{L} \leq 0.2$	Concentrated Load at Midspan
Simply-supported, or fixed both ends	$R_s = 2.5V_p$	$R_s = \frac{2V_p}{1 - \frac{d}{L}}$	$R_s = 2V_p$
Fixed one end and hinged one end	$R_s = 2.5(V_p - \frac{M_p^n}{L})$	$R_s = (\frac{2}{1-\frac{d}{L}})(V_p - \frac{M_p}{L})$	$R_s = 2(V_p - \frac{M_p^n}{L})$

Diagonal Tension Resistance:

$$r_s = 100 \left[\frac{1}{3} + \frac{1}{2} \frac{\varphi_{e\text{-}avg}}{\varphi_c}\right] (\varphi_c f'_c)^{1/2} \left(\frac{d}{L}\right)^2 \left(1 + \frac{2\varphi_v}{10^5} f_{dy}\right)$$

where φ_c = percent positive steel at midspan

$\varphi_{e\text{-}avg}$ = average of negative steel percentages at ends

φ_v = percent web reinforcement

Note: For $\frac{\varphi_{e\text{-}avg}}{\varphi_c} > 1.0$, use $\frac{\varphi_{e\text{-}avg}}{\varphi_c} = 1.0$

Stiffness:

	Uniform Load	Concentrated Load at Midspan
Simply supported,	$k = \frac{384E_c I}{5L^3}$	$k = \frac{48E_c I}{L^3}$
Fixed one end and hinged one end,	$k = \frac{160E_c I}{L^3}$	$k = \frac{106E_c I}{L^3}$
Fixed both ends,	$k = \frac{307E_c I}{L^3}$	$k = \frac{192E_c I}{L^3}$

For concrete, $E_c = 1000\ f'_c$

$$I = \frac{b(k'd)^3}{3} + \frac{nb\varphi d^3}{100}(1 - k')^2$$

$$n = \frac{E_s}{E_c} = \frac{30,000}{f'_c}$$

Values of k':

	$f'_c = 3000$	$f'_c = 5000$
$\varphi = 0.5$	0.27	0.22
1.0	0.36	0.29
1.5	0.42	0.34

Note: Yield deflection $= x_e = \frac{R}{k} = \frac{rbL}{k}$

Natural Period (Seconds):

Uniformly distributed mass:

Simply supported, $T = \frac{L^2}{42,500\ d\sqrt{\varphi}}$

Fixed one end and hinged one end, $T = \frac{L^2}{63,800\ d\sqrt{\varphi}}$

Fixed both ends, $T = \dfrac{L^2}{85{,}000\, d\sqrt{\varphi}}$

where L and d are in inches.

Note: If there is supported mass in addition to the beam or slab itself these periods should be increased by $\sqrt{M'/M}$ where M' is the total mass and M that of the beam or slab alone.

Concentrated mass at midspan:

Simply supported, $T = 0.91\sqrt{\dfrac{W_c L^3}{E_c I g}}$

Fixed one end and hinged one end, $T = 0.61\sqrt{\dfrac{W_c L^3}{E_c I g}}$

Fixed both ends, $T = 0.45\sqrt{\dfrac{W_c L^3}{E_c I g}}$

where I is computed for the transformed cross section.

5B.6 R/C TEE-BEAMS

Resistance: The resistance may be taken as the same as a rectangular beam having a width equal to that of the stem of the tee-beam.

Stiffness and Natural Period: The equations given for steel beams may be used with I taken as the moment of inertia of the transformed tee-section.

5B.7 TWO-WAY SLABS

Flexural Resistance:

$$r_f = 0.108\,(\varphi_{SC} + \varphi_{SE})\, f_{dy} \left(\frac{d}{L_s}\right)^2 \left[\alpha\,\frac{\varphi_{LC} + \varphi_{LE}}{\varphi_{SC} + \varphi_{SE}} + \frac{2 - \alpha}{3 - 2\alpha}\right]$$

where, φ_{SC} = percentage of bottom steel in short direction

φ_{SE} = percentage of top steel in short direction

where, φ_{LC} = percentage of bottom steel in long directio

φ_{LE} = percentage of top steel in long direction

L_S = shorter span

L_L = longer span

α = L_S/L_L

Shear Resistance:

The resistance of a two-way slab in pure shear or diagonal tension may be taken as $2(1+\alpha)/3$ times that for a one-way slab spanning the short direction when α is greater than 1/2 and the same as a one-way slab when α is less than 1/2

Stiffness:

	Simple Supports	Fixed Supports
$\alpha = 1.0$	$k = 252 \dfrac{E_c I}{L_S^3 L_L}$	$k = 810 \dfrac{E_c I}{L_S^3 L_L}$
= 0.9	= 230 "	= 742 "
= 0.8	= 212 "	= 705 "
= 0.7	= 201 "	= 692 "
= 0.6	= 197 "	= 724 "
= 0.5	= 201 "	= 806 "

where I = moment of inertia per unit width

Note: Yield deflection = $x_e = \dfrac{r}{k}$

Interpolate for 1, 2, or 3 sides fixed

Natural Period (Seconds):

$$T = 5.3\sqrt{\frac{w}{kg}} \qquad \text{simple supports}$$

where w = total weight per unit slab area

k = stiffness per unit slab area

$$T = 4.5\sqrt{\frac{w}{kg}} \qquad \text{fixed supports}$$

5B.8 FLAT SLABS

Flexural Resistance:

$$r_f = \frac{0.072}{K} f_{dy} (\varphi)\left(\frac{d}{L_L}\right)^2$$

where $K = \left[1 + \left(\frac{L_S}{L_L}\right)^2\right]\left[1 - \left(\frac{c_L}{L_L}\right)^2\right]\left[1 - \frac{3}{2}\frac{c_L}{L_L}\right]$

L_S = short span

L_L = long span

c_L = long column capital dimension

$\varphi = \varphi_{bL} + \varphi_{bS} + X(\varphi_{tL} + \varphi_{tS}) + X'(\varphi'_{tL} + \varphi'_{tS})$

φ_{bL} and φ_{bS} = average percentage of bottom steel in long and short directions

φ_{tL} and φ_{tS} = average percentage of top steel between drop panels in long and short directions

φ'_{tL} and φ'_{tS} = average percentage of top steel in drop panels in long and short directions

$X = 1 - p_L/L_L$

$X' = \frac{p_L}{L_L}\left(\frac{d_p}{d}\right)^2$

p_L = width of drop panel in long direction

d_p = depth to steel in drop panel

d = depth to steel in slab

Shear Resistance:

$$r_s = \frac{1}{A}\left[292\ bd + 0.04\frac{bd\ f'_c}{\varphi_o}\right]$$

where r_s = shear resistance, psi

A = area of panel outside column capital or drop panel

where b = perimeter of column capital or drop panel

$$\Psi_o = \frac{r_s}{r_f}$$

d = depth to steel in drop panel or slab

Stiffness (Interior panels):

c/L = 0.05	$k = 208 \ E_c I/L^4$
0.10	= 230 "
0.15	= 252 "
0.20	= 276 "
0.25	= 302 "

where I = moment of inertia per unit width (Trans. Sect.)

Note: L = average span

$$\text{Yield deflection} = \frac{r}{k}$$

Natural Period (Seconds):

$$T = 5.0\sqrt{\frac{w}{kg}}$$

where w = total weight per unit slab area

k = stiffness per unit slab area

5B.9 R/C DEEP BEAMS ($d/L > 0.4$)

Resistance:

May be computed by the same procedures given for other R/C beams.

Stiffness:

Simple supports,
$$k = \frac{1}{\frac{5L^3}{384E_cI} + \frac{L}{3E_cA}}$$

Fixed one end and hinged one end,
$$k = \frac{1}{\frac{L^3}{185E_cI} \ \frac{15L}{48E_cA}}$$

Fixed both ends, $$k = \frac{1}{\frac{L^3}{384E_cI} \quad \frac{L}{3E_cA}}$$

Where A = gross web area

<u>Natural Period</u>:

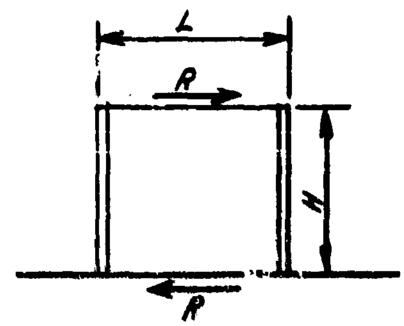
$$T = 5.5\sqrt{\frac{W}{kg}}$$

where W = total weight in span

k = stiffness as defined above

5B.10 R/C SHEAR WALLS

<u>Resistance</u>:



At cracking

$$R_c = 0.1\, f'_{dc}\, Lt$$

Ultimate

$$R_u = \frac{0.16}{\frac{P}{C} + 0.1} + 2.2P$$

$$\text{Where } C = A_{sc}\, f'_{dc}\left[15 + 1.9\left(\frac{L}{H}\right)^2\right]$$

A_{sc} = area of column steel on compression side

$P = A_{sv}\, f_{dy}$

A_{sv} = area of wall steel in each direction

<u>Stiffness</u>:

Prior to cracking:

$$k_c = \frac{E_c}{H\left[\frac{H^2}{3I} + \frac{2.2}{Lt}\right]}$$

where I = moment of inertia of uncracked section

After cracking:

$$k_u = k_c\,\frac{R_u}{R_c}\,\frac{L^2}{24H^2}$$

Natural Period (Seconds):

$$T = 2\pi\sqrt{\frac{W}{k_c g}}$$

where W = total weight as roof level plus 1/3 weight of walls

k_c = uncracked stiffness

5B.11 R/C Columns

Axially Loaded Columns:

Strength:

$$P_u = (0.85 f'_{dc} + \frac{\varphi_T}{100} f_{dy})\, bt$$

where φ_T = total steel percentage

b,t = cross-section dimensions

for $\frac{L}{t} > 15$, $P'_u = P_u(1.6 - 0.04\frac{L}{t})$

where L = unbraced length

Beam-Columns:

The strength of members subjected to combined bending and direct stress may be determined by use of Fig. 5A-10.2 (see APPENDIX 5A)

5B.12 BUILDING FRAMES (STEEL OR R/C) (ONE-STORY)

Lateral Resistance:

$$R = \frac{\Sigma M'_p}{h}$$

Where, $\Sigma M'_p$ = the sum of the bending strengths of the column sections or the connections at all column ends in the frame

h = story height

Lateral Stiffness:

Columns fixed both ends, $k = \frac{12E}{h^3}\Sigma(I)$

Columns fixed one end and hinged one end, $k = \frac{3E}{h^3}\Sigma(I)$

Where $\Sigma(I)$ = sum of all column moments of inertia

Natural Period (Seconds):

$$T = 2\pi\sqrt{\frac{W}{kg}}$$

where W = total weight of deck, frame, etc., at roof level plus 1/3 of weight of walls

k = stiffness as defined above.

5B.13 R/C ARCHES

Resistance:

Compression mode

$$r_c = (0.85\, f'_{dc} + \frac{\varphi_T}{100} f_{dy}) \frac{t}{r}$$

Where t = arch thickness

r = arch radius

r_c = resistance in psi

φ_T = total steel percentage

Flexural mode

$$r_f = 0.072\, \varphi f_{dy} \frac{d^2}{(\beta r)^2}$$

Where d = depth to tension steel

β = one-half the central arch angle

φ = steel percentage in one face

Buckling

$$r_b = (\frac{\pi^2}{\beta^2} - 1) \frac{E_c t^3}{12(r)^3}$$

Where r_b is the uniform pressure at which bucking occurs.

Natural Period (Seconds):

Compression mode

$$T = \frac{r}{1800}$$ where r = arch radius in feet

Flexural mode

$$T = \frac{\beta^2 (r)^2}{42{,}500\, d \sqrt{\varphi}} \cdot \psi \qquad \text{where } r \text{ and } d \text{ are in inches}$$

$$\psi = \frac{(\pi^2/\beta^2) + 1.5}{(\pi^2/\beta^2) - 1} \qquad \text{when cover at crown} = 0$$

ψ = should be taken as unity when cover at crown is greater than 0.1 times the span. Use linear interpolation between these two points.

These periods should be increased by $\sqrt{M'/M}$ when there is mass in addition to the arch itself.

5B.14 STEEL ARCHES

Resistance:

Compression mode

$$R_c^a = \frac{f_{dy} A}{r}$$

Where A = cross-section area

r = arch radius

Flexural mode

$$R_f^a = \frac{8 M_p}{(\beta r)^2}$$

Where β = one-half central arch angle

Buckling

$$R_b^a = \frac{\pi^2 EI}{\beta^2 (r)^3} \left(1 - \frac{\beta^2}{\pi^2}\right)$$

Where I = moment of inertia

Note: R^a is resistance per unit arc length

Natural Period (Seconds):

Compression mode:

$$T = 2\pi r \sqrt{\frac{w}{EAg}}$$

Where A = cross-section area

w = weight per unit arc length

r = radius

Flexural mode

$$T = 0.64\ (\beta r)^2 \sqrt{\frac{w}{EIg}} \cdot \psi$$

Where ψ is as defined in SECTION 5B.13

5B.15 R/C DOMES

Resistance:

Compression mode

$$r_c = (0.85\ f'_{dc} + \frac{\varphi_T}{100} f_{dy}) \frac{2t}{r}$$

Where t = dome thickness

r = dome radius

φ_T = total steel percentage in one direction

Flexural mode

$$r_f = (0.85\ f'_{dc} + \frac{\varphi_T}{100} f_{dy}) \frac{t}{r}$$

Buckling

$$r_b = 1.2\ \frac{E_c t^2}{(r)^2}$$

Natural Period (Seconds):

Compression and Flexural Mode

$$T = \frac{r}{2500}$$ where . is radius in feet

5B.16 COMPOSITE BEAMS

Flexural Strength:

$$M_p = A_B f_{dy} \left[\frac{d}{2} + t - \frac{A_B f_B}{2bf'_{dc}}\right] \approx A_B f_{dy} \left(\frac{d}{2} + \frac{t}{2}\right)$$

Where A_B = area of steel beam
d = depth of steel beam
t = slab thickness
b = effective slab width

Notes: Total slab compressive strength must not be less than $A_B f_{dy}$.

Required shear strength of concrete slab (on vertical section adjacent to beam) = $2A_B f_{dy}/tL$.

Required total strength of shear connectors in half span = $A_B f_{dy}$.

Resistance:

$$R = \frac{8M_p}{L} \text{ (simple span - uniform load)}$$

Stiffness:

$$k = \frac{384 E_s I}{5L^3} \text{ (simple span - uniform load)}$$

Where $E_s = 30 \times 10^6$ psi
I = transformed (to steel) moment of inertia of composite section

Natural Period:

Use same expression as for steel beams.

5B.17 FOUNDATION MATERIALS

Dynamic bearing pressures for the design of footings may be taken as follows:

For rock, the crushing strength

For granular soil, the bearing pressure which applied statically would produce a one-inch settlement.

For cohesive soil, three-quarters of the failure load.

If detailed soil information is lacking the allowable bearing pressure may be conservatively taken as twice the conventional allowable static pressure plus the side-on overpressure (p_{so}). In no case need the total footing area be greater than the roof area.

5B.18 COLUMN FOOTINGS

Resistance:

Flexure

$$r_f = \frac{7\varphi f_{dy} d^2}{100(L-a)^2} \quad \text{(square footings)}$$

Where r_f = resistance per unit bearing area
d = depth to tension steel
L = width of footing
a = width of column or base plate
φ = bottom steel percentage each way

Shear

$$r_s \cdot \frac{L^2 - (a + 2d)^2}{3.5\, ad\, f'_{dc}} = 0.035 + \frac{130}{f'_{dc}} + 0.07 \frac{r_f}{r_s}$$

Where r_s = shear resistance

5B.19 WALL FOOTINGS

Resistance:

Flexure

$$r_f = 0.0179\, \varphi f_{dy} \left(\frac{d}{l}\right)^2$$

Where r_f = resistance per unit bearing area
φ = percentage of bottom steel
l = footing projection outside wall

Shear

$$r_s = 45\left(\frac{d}{l}\right)^2 \left(\frac{\varphi}{2\varphi - \varphi'}\right)\sqrt{f'_c\, \varphi}$$

Where r_s = shear resistance per unit bearing area
φ = percentage of top steel

APPENDIX 5C. SUPPLEMENTARY MATERIAL RELATING TO EARTH SHOCK AND SHOCK MOUNTING

APPENDIX 5C. SUPPLEMENTARY MATERIAL RELATING TO EARTH SHOCK AND SHOCK MOUNTING

5C.1 INTRODUCTION

In this appendix are presented a number of items pertaining to eart shock and shock mounting which supplement the material given in Section 5.4. For a discussion of air-induced effects the reader is referred to Section 5.4. The material presented here includes additional considerations involving layered media, modifications for transeismic and subseismic conditions, direct-transmitted ground shock, combined random and systematic pulses, and comments on design of interior structures and equipment supports to resist ground shock motions.

Most of these topics are under active study, but the latest information and recommendations concerning these topics is presented as an aid in analyzing and designing for shock loading situations.

5C.2 NOTATION

The following notation is used in the expressions that follow.

a = maximum acceleration, in gravities

c = seismic velocity of medium, in ft. per sec.

d = maximum elastic component of transient displacement, in in.

E = Young's modulus of elasticity, in psi. For plane waves E is given by

$$E = \frac{(1 + \mu)(1 - 2\mu)}{(1 - \mu)} \rho c^2 \tag{5C-1}$$

where ρ is the mass density of the soil, μ is Poisson's ratio, and c is the seismic velocity as defined above. For values of μ of 0.25 or less, the relationship is approximately $E = \rho c^2$, and for soil with a density of about 115 lb. per cu. ft., an approximate value of E is

$$E = 25,000 \text{ psi } \left[\frac{c}{1000 \text{ fps}}\right]^2$$

f = frequency of system, in cps

p_{so} = peak overpressure in shock wave, in psi

p_i = incident stress at interface, in psi

p_r = reflected stress at interface, in psi

p_t = transmitted stress at interface, in psi

R = range from ground zero, in ft.

t_1 = effective duration of shock, in seconds, corresponding to a triangular pressure-time representation having the same positive phase impulse as the actual shock. For the derivation of this expression see Reference (1), SECTION 5C.8. The overpressure duration, t_1, may be expressed as follows:

$$t_1 \simeq 0.40 \text{ sec.} \left[\frac{100 \text{ psi}}{p_{so}}\right]^{0.6} \left[\frac{W}{1MT}\right]^{1/3} \qquad (5C\text{-}2)$$

$$(\text{for } p_{so} > 30 \text{ psi})$$

t_r = effective velocity pulse rise time, in sec., or $t_r \simeq \frac{1}{2}\frac{y}{c}$ for a homogeneous medium

T_o = positive phase of strain or velocity pulse, in sec., for direct-transmitted ground shock as defined in Fig. 5C-2.

T_r = rise time to maximum strain or velocity, in sec., for direct-transmitted ground shock as defined in Fig. 5C-2.

u = relative displacement of spring as defined in Fig. 5C-7

v = maximum velocity, in ft. per sec.

U = shock velocity, in ft. per sec.

$\overline{W}$ = yield of weapon, in lb. of TNT

W = yield of weapon, in kilotons or megatons, as noted

y = vertical depth below surface to point considered, in ft.

α = attenuation factor for velocity or stress

ϵ = strain, in./in.

ψ = $E_m c_n / E_n c_m$, where subscripts refer to adjacent layers.

5C.3 AIR-INDUCED FREE FIELD EFFECTS--LAYERED MEDIA

The layered media situation, which is complicated, was mentioned only briefly in SECTION 5.4. At present, for lack of a better method, it is convenient to use an approach that involves the step-wise passage of a stress wave downward through the medium. The procedure is illustrated in Fig. 5C-1 for a two-layered system where the transit times corresponding to passage of the stress wave down through the soil are as indicated in the figure. In general the basic concepts governing the computation of displacement, velocity and acceleration are the same as those described for a uniform medium. The displacements at any particular time may be computed by dividing the average pressure in an interval by the modulus of elasticity to get the strain, and multiplying by the length of the interval to get the displacement; the total displacement occurring over the length of the pulse is the sum of the displacements computed from great depth (point of zero stress) up to the point

considered. It is recommended that in this computation one assume no reduction in peak stress, no change in rise time, and a constant duration t_1.

The velocity is computed by means of the relations applying to homogeneous material (SECTION 5.4) having a value of c equal to that at the depth considered. The acceleration is computed by using the velocity at the point considered and a rise time equal to one-quarter the total transit time from the surface. It is recommended that the same ratios of horizontal to vertical effects be taken as for the homogeneous case, namely 1/3 for the horizontal displacement, 2/3 for velocity, and accelerations equal.

Complications arise at the interface of two media because of stress transmission and reflection. In soil and rock media the interface may not be sharply defined and the reflected and transmitted stress probably does not follow the laws governing purely elastic media. If it is known that the interface is fairly sharp, an estimate of the reflected and transmitted stresses can be made from the following relationships.

$$p_r = \frac{1 - \psi}{1 + \psi} p_i \tag{5C-3}$$

$$p_t = \frac{2}{1 + \psi} p_i$$

where ψ is the ratio of the impedances of the two media.

Reflected and transmitted pressures are illustrated by dotted lines in Fig. 5C-1. It should be noted that the stresses at the interface must be equal, and that from considerations of continuity the displacements must be equal. With care and judgment, it generally is possible to arrive at reasonable estimates of acceleration, velocity and displacements in layered media.

5C.4 TRANSEISMIC AND SUBSEISMIC

Generally it is advisable to compute all effects, (velocities and accelerations are of particular interest here,) as if superseismic conditions existed for a homogeneous case, or

$$U > c$$

For the pressure levels under consideration U may be computed from the relationship

$$U = 1130 \text{ fps} \sqrt{1 + \frac{6p_{so}}{7P_o}}, \quad \text{where } P_o \text{ is atmos. press.}$$

or approximately $$U \simeq 2800 \text{ fps} \sqrt{\frac{p_{so}}{100 \text{ psi}}} \quad \text{for } p_{so} > 100 \text{ psi}$$

Denote the quantity computed in this way by $\bar{Q}$. Then, modify $\bar{Q}$ to obtain the desired quantity Q by multiplying by the factor β as indicated below.

Condition	Value of $\beta = Q/\bar{Q}$
$c < U$	1.0
$U < c < 1.5U$	c/U
$1.5U < c < 2U$	1.5
$2U < c < \infty$	$1 + \frac{U}{c}$

It is noted that this multiplies the computed velocity and acceleration value by a factor having a maximum value of 1.5 at a seismic velocity such that the air shock velocity is nearly equal to the Rayleigh wave velocity. Displacements do not appear to be affected by the transitions under consideration, and for lack of better information at present it is recommended that deflections not be modified by the noted coefficients.

The question naturally arises as to which value of c to use in a layered system. It is tentatively recommended that each layer be considered separately with its appropriate value of c, but if lower layers give a higher value of acceleration or velocity, use the larger value at all higher levels, or as a base for obtaining values at higher levels.

5C.5 DIRECT-TRANSMITTED GROUNDSHOCK--SURFACE BURST

The energy transmitted directly to the earth from a surface or near surface burst can be propagated effectively through sound material for long distances. Experimental data of this nature are available only for buried HE shots, and in a few cases for very small nuclear weapons [References 5C(2), 5C(3) and 5C(4)]. Studies of these data and of some of the preliminary theoretical work in progress have led to the relationships presented. The estimates of acceleration, velocity and displacement given herein may be conservative, but probably are not unduly so.

For a completely buried shot, the first portion of the strain or velocity record at a distance R from the point of burst has the form shown in Fig. 5C-2. The corresponding acceleration and displacement wave forms also are shown in the figure. The relative magnitudes of the times shown in Fig. 5C-2 are taken from data given in Chapter VII of Ref. 5C(2).

Without serious error the relation between peak strain and the peak particle velocity v is given by

$$v = \frac{c\ p_{so}}{E} \qquad (5C\text{-}4)$$

Assuming that the steepest part of the velocity-time curve has a slope twice the average slope during the rise phase leads to the following relationship

between peak acceleration and peak velocity

$$a = \frac{2v}{T_r} \simeq \frac{12c^2}{R} \epsilon \tag{5C-5}$$

An estimate of the maximum displacement d may be obtained by computing the area under the positive phase of the velocity curve of Fig. 5C-2. If the area shown is about the same as that represented by a parabola, then

$$d = \frac{2}{3} v T_o \simeq \frac{1}{3} R\epsilon \tag{5C-6}$$

If data for strain, acceleration or displacement are available, approximate relations for the other quantities may be obtained. In general the seismic velocity enters into the relationships. For fully buried detonations experimental data indicate that in general the strains are independent of c, the velocities depend on c, and the accelerations, stresses and energy on c^2. However, some available data for very small charges indicate other relationships possibly may exist.

With respect to energy partition, available data indicate a net effectiveness in terms of yield of surface nuclear compared with buried HE of approximately 0.01 and this figure is used herein in relating the effects of the two conditions.

The relationships developed below are restricted to granite for surface bursts and more specifically to ranges where the overpressure at the surface is between 100 and 600 psi.

Data are available for acceleration from RAINIER in volcanic tuff with a seismic velocity of 6000 fps [Ref. 5C(4)]. In the range of interest, scaled in terms of megaton weapons and seismic velocity, the acceleration may be expressed as follows:

$$a = 0.36g \left[\frac{W}{1MT}\right]^{5/6} \left[\frac{1000\ ft}{R}\right]^{3.5} \left[\frac{c}{1000\ fps}\right]^2 \tag{5C-7}$$

From Eq. (5C-4), (5C-6) and (5C-7) the following may be obtained:

$$v = 0.95\ fps \left[\frac{W}{1MT}\right]^{5/6} \left[\frac{1000\ ft}{R}\right]^{2.5} \left[\frac{c}{1000\ fps}\right] \tag{5C-8}$$

$$d = 3.8\ in. \left[\frac{W}{1MT}\right]^{5/6} \left[\frac{1000\ ft}{R}\right]^{1.5} \tag{5C-9}$$

For slant ranges it is recommended that these be considered as horizontal components. The vertical components of displacement and strain

may be taken one-half as great, but the velocity and acceleration should be taken the same. It is emphasized that the derived relationships are for surface bursts in a homogeneous medium, are intended to be used only in the range corresponding to overpressures of 100 to 600 psi, and are derived solely for granite material. However, in lieu of other data, these relations may be used for other materials and conditions. It is recommended that in general the seismic velocity for granite deposits be taken not greater than 12,000 fps; fissuring and other factors tend to give a gross seismic velocity for the deposit as a whole which is usually much less than laboratory tests give for small samples.

5C.6 DIRECT-TRANSMITTED GROUND SHOCK--LAYERED SYSTEMS

For estimating the velocities and accelerations only, not displacements, in a two-layered system, the method illustrated in Fig. 5C-3 based on ray-paths may be used as a best approximation at present. The principle used is based on an effective value of c, designated by $\bar{c}$, for which the transit time by a direct wave from the source to the target is the same as for the fastest transit time of a shock wave in the complex layered system.

Reference to Fig. 5C-3 will show the following relationships to exist.

$$\sin \varphi = \frac{c_1}{c_2}$$

and

$$\frac{R}{\bar{c}} = \frac{(2H - y)\sec\varphi}{c_1} + \frac{R - (2H - y)\tan\varphi}{c_2}$$

Thus

$$\frac{c_1}{\bar{c}} = \frac{c_1}{c_2} + \frac{2H - y}{R}\sqrt{1 - \left(\frac{c_1}{c_2}\right)^2} \qquad (5C\text{-}10)$$

It will be noted that if

$$\frac{2H - y}{R} \longrightarrow 0, \quad \bar{c} \longrightarrow c_2$$

and if

$$c_1 = c_2 \ , \quad \bar{c} = c_1$$

As an example, if $H = 100$ ft., $y = 60$ ft., $R = 2000$ ft., $c_1 = 2000$ fps and $c_2 = 8000$ fps, then one finds from Eq. (5C-10) that $\bar{c} = 6300$ fps.

5C.7 COMBINED RANDOM AND SYSTEMATIC PULSES

Ordinarily the input for ground motion consists of two parts, a systematic portion on top of which is superimposed a series of random oscillations. The magnitude of the peaks of the random components may be either small or large compared to the systematic portion. The random part may exist over the entire range of the systematic portion, only part of the range, or even prior to the systematic portion. Schematically the parts may be related as in Fig. 5C-4 where the actual input at any time is the sum of the two input curves.

For a random series of pulses, the relative velocity peak of the spectrum compared with the maximum input velocity can be high, but is unlikely to be much higher than about 3 to 5 unless an almost resonant condition is obtained with several pulses of alternate positive and negative sign of exactly the same shape and duration. Such a resonant condition for velocity is extremely unlikely from blast loading, and has not been observed even in earthquake phenomena. Even if for some reason partial resonance is achieved, damping will reduce the peaks considerably.

In general the combined effect of the two input motions, systematic and random, depends on their individual effects. In Fig. 5C-5 are shown sketches of the response curves corresponding to each of the parts of Fig. 5C-4. The response spectrum corresponding to the systematic component (a) is a relatively sharp-peaked, nearly-triangular curve and generally with the peak at a relatively low frequency, while the response spectrum corresponding to input (b), the random component, is flatter and broader. The combination of the two spectra will be roughly of the same general shape as (b) but with a longer base. There may be a higher peak as well.

It can be shown rigorously that the combined spectrum will in all cases be either equal to or less than the sum of the spectra corresponding to the individual inputs. In general, although it has not been rigorously proven, it appears reasonable that the combined spectrum can be expected to be equal approximately to the square root of the sums of the squares of the individual spectra, point by point. In most practical cases of the type under consideration, because of the fact that the frequencies for which the spectrum values are important differ by a considerable amount, the sums of the spectra or the square root of the sums of the squares are nearly the same as the maximum individual modal value.

Until such time as better information becomes available it is recommended that the bounds for design shock spectra, as discussed in SECTION 5.4.1, be taken in accordance with the values presented in Table 5C-1. Normally, for air-induced shock in homogeneous media and superseismic conditions, the bounds corresponding to a simple pulse and high damping would be applicable; for layered media and superseismic conditions, and all transseismic situations, the bounds corresponding to a complex oscillatory pulse and high damping are applicable. For all cases involving direct-transmitted shock it is recommended that the bounds corresponding to a complex oscillatory pulse and high damping be used.

5C.8 COMMENTS ON DESIGN TO RESIST GROUND SHOCK MOTIONS

(a) Introduction. The comments contained herein concern the design of underground installations to resist the ground shock from high overpressures in accordance with the estimates of input ground motion made in APPENDIX 5C and SECTION 5.4.

Whether the structure be a shallow box, arch, or a deep underground structure, the input motions and the response spectra corresponding thereto, for the free-field motions, are used in the same way, and no distinction is made here between these structures. The primary consideration given here is to the type of interior structure which consists of a two-to-four story building frame supported independently of the roof covering, so that the base motion to which the frame is subjected corresponds in many respects to earthquake base motions. However, many of the comments regarding design of equipment are pertinent to the situation where equipment is mounted directly on a box structure without an independent interior frame.

(b) Comments on Shock Mounting. Equipment in a structure subjected to shock is forced to respond in a manner determined by the structural behavior. The shock motion of the foundation of the building is assumed to be known, corresponding to some relatively simple motion (possibly a single sine curve of displacement) on which is superimposed a random pattern of relatively higher acceleration pulses with only a small amplitude of motion. The net effects of the ground motion are most readily described in terms of a response spectrum such as the one shown in Fig. 5C-6. The three straight line bounds for the spectrum, shown as A, B, and C, are determined as follows:

A corresponds to the maximum ground displacement (taken in the example as 5 in.);

B corresponds to 1.5 times the maximum ground velocity (taken as 33.3 in. per sec., hence the line is drawn at 50 in. per sec.);

C corresponds to the maximum ground acceleration (taken at 5.0g).

A single-degree-of-freedom elastic system, subjected at its base to the input motions described, would suffer a maximum relative motion of the supported mass to the base corresponding to the frequency of the system. If the frequency were 1.0 cycle per sec. the relative maximum displacement would be 5 in. (or the same as the base motion, in this case) and the absolute maximum acceleration would be 0.50g. However, if the supported system had a natural frequency of 3.0 cycles per sec., the maximum relative displacement would be 2.6 in., and the maximum acceleration would be nearly 2.5g. If the mass could not withstand an acceleration greater than 1.0g, it would be necessary to soften the spring supports to the extent that the frequency would be less than 1.5 cycles per second.

However, other alternatives are possible, including permitting yielding of the "spring" supporting the mass. A large structure, say a multi-story frame, has several degrees of freedom and can oscillate or respond to shock in several different modes. It is possible to relate the response of such a structure to that of a single-degree-of-freedom structure. [See SECTION 5C.8(j)]

Each piece of equipment in the structure is itself a dynamical system, and interacts with the structure to form a highly complex system. When the designer modifies the structure to enable it to withstand the shock motions, he affects the equipment in the structure also. It may be necessary in the design of the structure to provide flexibility or energy absorption in order to permit survival of the equipment in the structure, particularly if the equipment is highly shock sensitive.

The particular structure referred to herein is a three or four story frame subjected to quite violent motions of the base. Such a structure can be "shock-mounted" as a whole by soft springs at the base, or individual floors can be isolated. As an alternative, each sensitive piece of equipment can be separately isolated. However, it is likely that the structure itself, with only simple precautions and provisions allowed for in the structural design, can provide a sufficient degree of attenuation of the shock motions transmitted to the equipment that in most cases no further isolation is required, except possibly for vertical motion.

(c) Effect of Plastic Behavior in a Simple System. Consider the simple system composed of a mass m and a spring having a spring constant k in the elastic range, as shown in Fig. 5C-7, where the relation between force in the spring and relative deflection or strain is as indicated in Fig. 5C-8. The base is subjected to a disturbance x. When the structure behaves elastically, if the maximum input motion, velocity, and acceleration have magnitudes corresponding to those used in drawing Fig. 5C-6, the maximum responses of the system can be read from the chart in Fig. 5C-6.

If, however, the spring becomes plastic at a value of relative displacement u_e, then the question arises as to the maximum responses generated for the same input motions x. The studies that have been made indicate that in general nearly the same relative displacements are reached, and that the same displacement response spectrum can be used, as a reasonable approximation. However, one cannot now interpret the same spectrum lines in terms of acceleration, in the region where the displacement exceeds the elastic limit value. In that region, only for computing acceleration, one must replace the part of the response spectrum lying above the relative displacement value u_e by a line parallel to line A, along the line $u = u_e$, where u_e corresponds to the elastic limit deflection as shown in Fig. 5C-8. Such a line, marked A_y, is drawn in Fig. 5C-6 corresponding to a value of yield displacement of 1 in. From Fig. 5C-6, it appears that a system having a frequency of 3.0 cycles per sec. would have a relative displacement of 2.6 in., for both elastic and elasto-plastic conditions, but would have an acceleration of nearly 2.5g for elastic conditions, and nearly 1.0 g for elasto-plastic conditions.

If now a very light piece of equipment is mounted on the mass M in Fig. 5C-7, and if the equipment mass is so small that it does not affect the response of M, in general the equipment acts like a system subjected to a revised base motion which is the motion of the mass M. However, now we need the absolute motion of M, which we cannot obtain conveniently from Fig. 5C-6. We can infer that the maximum base motion can range from a minimum value of $x_m - u_m$ to a maximum value of $x_m + u_m$, but this range is probably too large to be useful.

Nevertheless, a value of maximum acceleration can be obtained from Fig. 5C-6, and this value provides an upper limit of acceleration for the equipment. Under the worst possible conditions the response of mass M, which is the base of the equipment, will be a simple harmonic motion of magnitude corresponding to the maximum acceleration determined from the response spectrum in Fig. 5C-6. If the equipment is subjected to such an input, its response is a function of the ratio of the equipment frequency to the frequency of the motion of mass M.

The ratio of the equipment acceleration a_e to the "structure" acceleration a_s is given by the expression

$$\frac{a_e}{a_s} = \frac{1}{1 - f_s^2/f_e^2} \qquad (5C\text{-}11)$$

where f_e = frequency of the equipment,

f_s = frequency of the structure.

This relation is not valid when f_e is nearly equal to f_s. However, it indicates that the acceleration of the equipment will be less than twice that of the structure (or of mass M) when the frequency of the equipment is more than 1.4 times that of the structure, or less than about 0.8 times that of the structure. In other words, high acceleration in the equipment can be avoided by not "tuning" the equipment to the same frequency as the structure.

For example, in the illustration used previously, for a structural frequency of 5 cps and the response spectrum of Fig. 5C-6, the equipment acceleration will be less than 5g if the structure is elastic, or less than 2g if the structure has a yield deflection of 1 in., provided that the equipment keeps away from a frequency range between 2.4 and 4.2 cps. This frequency range may be avoided by appropriate shock mounting of the equipment, or it may have been implicitly avoided by the very nature of the equipment itself.

In summary, a design spectrum for a constant value of μ, the ratio of the total displacement to the elastic limit displacement, is, for maximum deflection, the original spectrum, but, for acceleration, it is a polygon drawn for values $1/\mu$ times the deflection values on <u>all</u> <u>three</u> <u>sides</u>. A structural design spectrum for $\mu = 5$ is shown in Fig. 5C-6 by the lines A_y, B', and C'.

(d) Use of Response Spectra for Multi-Story Buildings--Horizontal Motions. The use of the single-degree-of-freedom response spectrum for a multi-story building represents an approximation which requires study. The background of experience with earthquake resistant design indicates that the approximation can be useful. Rigorous use of the spectrum concept is possible only by consideration of the individual modes of dynamic response of the multi-story structure. A method for the elastic analysis of the complex system is briefly described in SECTION 5C.8(j).

As has been pointed out there are several reasons why the design spectrum can be lower in value than the response spectrum from the input motions. First of all, before failure of a building frame occurs there will be plastic action developed, and the responses will be affected thereby because of the energy absorption. There is still another influence about which too little is yet known. This concerns the interaction of relatively heavy structures with the ground when the ground motion occurs. Although the interaction is relatively slight, calculations that have been made in a preliminary fashion indicate that there is a series of peaks and valleys in the response spectra and the valleys actually correspond to the true response for the actual structure for which the input motions are measured whereas the peaks correspond to responses of structures with slightly different physical properties.

In the light of the above discussion it is believed reasonably conservative to use as a basis for the design a design spectrum plotted in the same way as the response spectra in Fig. 5C-6, with envelopes determined on the basis of the amount of plastic deformation that is permissible.

Insofar as a building frame itself is concerned, structural design can be accomplished with a reasonable degree of conservatism, by using the design recommendations proposed for earthquake design [Ref. 5C(5)]. In general, for building frames, these consist of two parts: (1) a specification on the base shear for which the design should be made; and (2), a specification for the design force distribution over the height of the building frame.

The force distribution over the height of the building corresponds to a linear distribution of acceleration ranging from zero at the base to a maximum at the top of the building. The accelerations corresponding to the design force distribution are shown in Fig. 5C-9. In the figure, there is given a derivation of the equations for force distribution given in Refs. 5C(5) and 5C(6) in which this magnitude of the force at any elevation is given by the relationship:

$$\frac{F}{V} = \frac{Wh}{\Sigma\, Wh} \qquad (5C\text{-}12)$$

in which F = lateral force at any height h above the base, corresponding to the mass of the building or weight of the building at that height.

V = total lateral design shear at the base.

W = the weight at the height h.

h = the height above the base of the building.

For a uniform building, the local force at the top of the building would correspond to an acceleration of 60 percent g, if the average base acceleration used in the design is 30 percent g.

From the design spectrum which has been modified to account for inelastic action one can read off the accelerations and therefore the seismic coefficients to be applied to the total weight of the building to determine its base shear. The base shear so obtained is then used with the procedure outlined in Fig. 5C-9, to determine the design shear distribution over the height of the building. These are considered along with the dynamic yield stresses of the material, in arriving at the final design magnitudes.

In using the spectrum for a multi-story building, the period of the building, or the frequency that corresponds to this period, for the lowest mode is used. Although a more accurate analysis taking account of the modal deformation of the building, in accordance with the methods described in SECTION 5C.8(j), can be made, such an analysis is not warranted in the light of present knowledge, for the building frame design itself. Moreover, it presents difficulties in that such a design cannot be completed until the parameters entering into it are known, and these are not known until the design is completed. Until methods are available for a more accurate study of the energy absorbing mechanisms, to lead to a rational basis for the reduction in effect that is found in practice, a modal analysis procedure must be applied with some caution in order to avoid excessively large magnitudes of the design forces. The procedures described in Ref. 5C(7) may be used, but this may be unduly involved for the problem at hand. The method recommended herein is probably more accurate, if plastic deformation in the lower story develops, than any other simple procedure.

From a slightly different point of view the structure can be considered as a series of masses and springs in line, with the spring flexibilities corresponding to the lateral flexibility of the columns between floors, and the masses being the floor masses. It can be seen that the lower columns, below the first floor, have a function corresponding to an isolating spring, and the more flexible they are, the better the shock isolation. The fundamental frequency of the complex structure takes account of the effective flexibility of the first floor columns and gives a basis for use of the simple spectrum of Fig. 5C-6 to represent the complex structural response.

For a uniform shear beam, of which the multi-story frame is an approximation, the first few frequencies are in the ratios 1:3:5:7, etc. The ratios are slightly different for a frame but not markedly so. The modal coefficients, adjusted for the participation factors for the modes, are roughly 1/2, 1/3, 1/6 -, and nearly zero for the last few modes in order. The sum is 1.0.

Hence it can be proved that the maximum relative displacement (or maximum acceleration) of the first floor is about the same as that in the spectrum of Fig. 5C-6, or less, corresponding to the fundamental frequency. It is the same if all frequencies lie along line A in Fig. 5C-6, it is about 2/3 if they all lie along line B, and about 1/2 if they all lie along line C.

The important fact is that for the first floor we can use the response spectrum of Fig. 5C-6 to determine an upper limit of maximum horizontal acceleration, in the elastic range.

For higher floors the acceleration and the displacement relative to ground are higher. The precise values depend on the framing and the structural parameters; the ratios for the top floor for a system with equal floor masses and equal column flexibilities throughout indicate the following results as an approximation:

All frequencies intersect spectrum on line	Relative displacements or accelerations compared with Fig. 5C-6		
	First Floor	Top story of 3	Top story of 4
A	1.0	1.6	1.7
B	0.7	1.3	1.4
C	0.5	1.2	1.3

In the absence of analytical data, a linear interpolation for intermediate stories may be used, although it is recommended that before doing so the first floor values be increased to 1.0 for all ranges.

For the design of the columns in the upper stories the procedure previously described and which is used for earthquake design sould be employed. This involves the base shear determination from Fig. 5C-6 and a distribution of lateral force over the height of the structure corresponding to a linear variation of acceleration, from a base value of zero to a top value of twice the average.

Calculations made for the effect of plastic action indicate the following results:

(1) Plastic behavior does not appreciably affect the maximum displacements of any story relative to ground.
(2) It will change accelerations in the same way as for a single-degree-of-freedom system. Therefore Fig. 5C-6 may be used for design in the elasto-plastic range.

The above comments apply whether the plastic behavior of the frame is due to yielding in the columns or in the girders. It appears preferable to cause yielding to occur in the girders rather than in columns. However, the yield moments at the ends of the girders must be sufficient to cause the appropriate limit design load of the frame to be reached.

The remarks made above also apply primarily to a structure in which the lower or base columns are designed to be of about the same flexibility as the columns in the first story. If the columns are hinged at the footings, and if they are about half as long, at least, as the distances between the

upper floors, this condition will be satisfied. It probably will be satisfied even if the columns are not hinged, but then higher lateral accelerations can be transmitted to the first floor, and this may not be satisfactory.

(e) Multi-Story Buildings--Vertical Direction. In the vertical direction the situation is more complex. The high frequency of the building in the vertical direction, particularly in the vertical oscillation of the columns, makes it possible for the forces to be transmitted almost directly vertically through the building to the beams. The beams will then oscillate as systems having a frequency corresponding to their frequency when partly fixed at the ends or simply supported, depending upon their connections, in accordance with their own mass and that of the weight which they carry. The design can be made then with the same type of modification of the spectrum as used for the horizontal direction, but without the provision for a distribution of shear over the height, because at the various elevations the responses will be roughly the same.

Because the blast shock in the vertical direction may be greater than in the horizontal direction, there may be a necessity for investigating more carefully the vertical effects on the building. However, some brief study of this problem will indicate that ordinarily, unless the design accelerations are quite large, this will be unnecessary.

For vertical loads, the ordinary design condition used will be approximately valid at dynamic yielding for a load corresponding to

$$2.2(DL+LL) \tag{5C-13}$$

in which DL is the dead load magnitude measured in terms of force per unit of area, and LL is the live load magnitude measured in the same way. The factor 2.2 comes from the ratio between the dynamic yield stress of approximately 44,000 psi to the design stress of approximately 20,000 psi. For static yield values the factor would be only 1.65. With a redistribution of moments corresponding to limit loading conditions, the factor may be increased to as much as 2.5.

For a vertical acceleration of Ng, the design must be made for a downward load of N times the weight, plus the weight itself, or, in effect, a load as follows:

$$(N + 1)(DL + FL) \tag{5C-14}$$

in which FL is the "fixed" live load or the live load actually in existence rather than the design live load. This might be taken as a sort of average value, because the local values are not of as great importance in determining the stresses as the average over-all value that actually is in place at the time of the shock.

The larger of these two relationships, in Eq. (5C-13) and (5C-14), governs the design. If the fixed live load, FL, is equal to the design live load, LL, then in order for the dynamic effect to govern, the critical

acceleration factor N must be greater than 1.2. But in general, FL is less than LL and N must be even larger. For FL = 0.5 LL, and LL = 2DL, which are reasonable average values, then the static design corresponds to a magnitude of 6.6 DL and the dynamic vertical design would correspond to (N + 1) 2 DL, in which case, in order for the dynamic vertical design to govern, the factor N must be greater than 2.3. In general it will be seen that only in rare circumstances or for extremely high accelerations, will it be necessary to take into account the vertical dynamic effect if the design is made under the usual static requirements for the dead load and live load effects.

If it is necessary to force the beams to go into the plastic range in order to achieve a sufficient degree of shock isolation for the equipment mounted on them, it does not appear feasible by design to limit the superimposed vertical accelerations to values of much less than about 2 to 3 g. Further reductions can be achieved only by spring mounting whole floors or by separate attention to the equipment.

Within several feet (approximately 2 times the depth of the beam) from the supports at the columns, the vertical accelerations are not appreciably attenuated by the elastic or plastic flexibility of the beam.. Either invulnerable equipment must be used near the columns, equipment placed in these regions must be shock mounted separately, or the entire structure must be shock mounted for vertical motion, to limit the accelerations near the beam supports.

(f) <u>Shock Mounting of Entire Structure</u>. When the entire structure is shock mounted, in essence the system is supported on a spring interposed between the base and the first mass of the structure. For horizontal motions, the column is already such a spring and probably no further springing is necessary. For vertical motions there may be some reason to consider additional isolation. In either case, it is desirable that the fundamental frequency of the shock mounted structure differ by at least a factor of 1.4 or more (preferably 2), from the frequency of the structure above the mounting, considered as on a fixed base. For example, a vertical frequency of the entire structure of 2 or 2.5 cycles per sec. will not cause resonance with the beams if the latter have a frequency of 4 cps. However, the isolation achieved may not be as great as permitting plastic action in the beams.

As a very rough approximation, the effects can be accounted for by use of Fig. 5C-6, for the new fundamental frequency of the shock mounted structure, and a multiplication factor may be used to account for the effect on the uppermost mass. For top floor for lateral motion 2.0, and for beams for vertical motion, 1.5 are suggested values. However, computations are desirable for these in particular cases.

(g) <u>Response of Light Equipment Mounted on Building Frame Members</u>. The situation is different for equipment than for the building frame. For the building frame, we can take into account the inelastic energy absorption. For the equipment, it may not be possible to do this. Consequently the actual spectrum values as given in Fig. 5C-6 may have to be used for the

design of the equipment, unless yielding shock mounts or other expedients are used to cut down the design values. In general, for elastic shock mounts, the influence is roughly given by a change in the frequency, and the results can be obtained by a shifting to the lower corresponding frequency on the shock spectrum curve.

For equipment mounted on the bottom floor, if the floor is supported directly on the rock, or for equipment near points of support such as columns (for vertical motion), the equipment will be subjected to the same intensities of input motion as the base, and the response spectrum should be used directly for the equipment. However, if the equipment is mounted on interior elements which are themselves flexible or which may become plastic, the response spectrum of the equipment may be modified because the equipment base is now subjected to a revised input motion.

Only preliminary studies are available for this problem. They indicate that for frequencies outside the range in which the structure or structural element becomes plastic the part of the structure which acts as a base for the equipment responds in the same manner as if the structure remained elastic. However, this motion is now different from the original structural base motion, and the response spectrum of the equipment is thereby affected.

Some data from submarine shock response indicates that for equipment mounted on bulkheads and partial bulkheads, the response spectrum may have ordinates of the order of 1.5 to 2 times the ordinates for the parts of the hull directly subjected to shock motion. This comes about because of the change in input motion.

At the present time it is not possible to develop a completely rational design procedure for the equipment without a complete analysis of the system consisting of the structure and the equipment. It is recommended therefore, that the following procedure be used until further data become available:

(1) For equipment mounted in an elastic structure not completely shock mounted, design the equipment, and its individual shock mounts if required, for the basic shock spectrum for vertical or horizontal motion.

(2) For equipment mounted in a structure not completely shock mounted, but designed to become plastic in such a way that only a reduced force can get through the plastic element, the acceleration transmitted to the equipment "isolated" by the plastic element cannot exceed that corresponding to the plastic strength of the element of the structure which becomes plastic. In other words, the equipment can possibly be designed for the "design" spectrum instead of the response spectrum. However, in order to avoid secondary effects, it is recommended that the equipment be designed for twice the "design" spectrum but not for a value exceeding the response spectrum.

(3) For a completely shock mounted structure, further complications arise, and additional analysis may be required. Tentatively it is recommended that the equipment be designed for twice the maximum acceleration which the entire structure sees, and for the maximum velocity and maximum displacement in the structural response spectrum. However, if frequencies of the equipment approach those of the entire shock mounted structure resonance may occur. Frequencies between 1/2 and 2 times those of the structure must be avoided, or provision made for them by considering a resonance phenomenon with a sustained harmonic input. This has been discussed in SECTION 5C.8(c).

(h) Problems in determining Response of Heavy Equipment. The procedures described above for determining the response of light equipment are not unreasonable, although in some circumstances the response may be even greater because of resonance. One should avoid particularly a frequency of the equipment equal to the frequency of the member on which it is mounted. When the equipment is heavy, however, there is a feedback mechanism in which the response is less than it would be by the methods described above. This problem is under study. No definitive analytical means are yet available for handling the problem in a simple fashion. Consequently, it is recommended that the same procedures be used as for light equipment, although it is recognized that such procedures may be overconservative. In special cases, an analysis can be made of the actual system.

(i) Equipment Vulnerability and Design Recommendations. The determination of the vulnerability of equipment to shock is a very difficult problem. It is not sufficient to state an acceleration limit, as the frequency corresponding to this limit is also a factor. In general a vulnerability spectrum can be drawn as a function of some measure of frequency of the input motion. This will have peaks at the natural frequencies of the piece of equipment. If these are close together, possibly a uniform acceleration vulnerability may be postulated, although this probably drops down for low frequency inputs.

In any event, the vulnerability we are concerned with is that due essentially to a single pulse for low frequency inputs, or to several pulses for high frequency inputs, but certainly not that due to a steady state oscillatory input. In any case, it appears that we need not be generally concerned with inputs having a frequency higher than about 15 to 20 cps for vertical motion, except near the columns, and about [illegible] cps, even for the highest mode of lateral motion, for horizontal motion. The former frequency can be reduced if the beams are reduced in section near their ends, or supported on the longitudinal girders.

If the structural design does not achieve the necessary degree of attenuation of acceleration, then the equipment may be shock mounted, or the design modified. It is usually cheaper and simpler to shock mount the equipment except in very special cases. It appears entirely feasible to limit the shock accelerations experienced by equipment to about 2.5 g for vertical motion, and possibly about 1.5 g for horizontal motion, by appropriate measures

in the structural design. Any further reductions can be achieved only by unusual methods, and require more detailed study and analysis. It is recommended that further reductions, if needed, be obtained by individually shock mounting vulnerable pieces of equipment. It is not clear at this time whether any piece of equipment is in fact sensitive to less than 1.5 g for the actual type of motion to which it will be subjected. It is possible that higher frequency steady sinusoidal motions may cause damage to equipment at lower accelerations, but this is not pertinent to the problem.

In order to reduce somewhat the accelerations near the columns, relatively thin energy absorbing pads may be used. These will not achieve a major shock isolation effect, but they will be of help in keeping high frequencies from being transmitted through the columns.

(j) Outline of General Theoretical Approach for Multi-Degree-of-Freedom Linear Systems. In general, the analysis for a multi-degree-of-freedom system subjected to blast shock can be made analytically with a procedure which involves a number of steps as follows.

(1) For the complex system, find the modes and frequencies. For each mode find the stress at the point considered or the quantity at the point which is desired.

(2) If the system is one which is subjected only to base motion, find the excitation coefficient for each mode. This is defined as the expansion of a unit deflection of all the masses, in the direction of the base motion, into a series of modal deflection shapes. The excitation coefficient is the coefficient of the particular modal shape in this expansion. For other kinds of input motion, the excitation coefficients have to be defined in a different fashion. This is not discussed here.

(3) Now determine the response spectrum for the quantity desired for a single-degree-of-freedom system.

(4) The modal response is then determined as the product of the stress or particular response in each mode, times the excitation coefficient for that mode, times the response spectrum value for the frequency of the mode.

(5) The maximum response of the system for the particular response quantity that is desired is less than the sum of the modal maxima.

(6) For a system with several degrees of freedom, the actual maximum response will not ordinarily exceed greatly the square root of the sums of the squares of the modal responses. Even in a two-degree-of-freedom system the excess will be less than thirty percent. Consequently, the root mean square value can be used as a design basis rather than the sum of the modal maxima, particularly where the number of modes is three or greater.

5C.9 REFERENCES

5C(1) Newmark, N. M., and Hall, W. J., "Preliminary Design Methods for Underground Protective Structures," Report prepared under Contract AF 29-(601)-1171 for the Air Force Special Weapons Center, Kirtland Air Force Base, New Mexico, December 1959, AFSWC-TR-60-5. (Secret).

5C(2) "An Engineering Manual on Design of Underground Installations in Rock," prepared by U. S. Bureau of Mines for the Corps of Engineers, draft copy, March 1957 (Confidential).

5C(3) Engineering Research Associates, "Underground Explosion Test Program, Final Report, Vol. II, Rock," 30 April 1958 (Confidential).

5C(4) Perret, W. R., and Preston, R. G., "Preliminary Summary Report of Strong-Motion Measurements from a Confined Underground Nuclear Detonation," ITR-1499, Operation PLUMBBOB. June 15, 1958 (Unclassified)

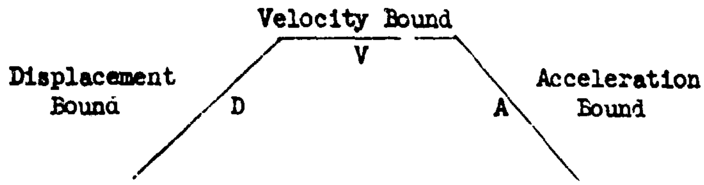
5C(5) Structural Eng'rs Association of California, "Recommended Lateral Force Requirements, Oct. 1958 (Unclassified).

5C(6) Anderson, A. W., Blume, J. A., et al, "Lateral Forces of Earthquake and Wind," Trans. ASCE, Vol. 117, 1952, p. 716-780 (Unclassified).

5C(7) Zeevaert, L., and Newmark, N. M., "Aseismic Design of Latino-Americano Tower in Mexico City," Proceedings World Conference on Earthquake Engineering, Earthquake Engineering Research Institute, Berkeley, California, 1956, p. 35-1 -- 35-11 (Unclassified).

TABLE 5C-1

DESIGN SHOCK SPECTRUM BOUNDS



Type of Input —	Simple Pulse		Complex Oscillatory Pulse	
Amount of Damping —	low	high	low	high
D/d	1	1	2 to 3	1
V/v	1.7	1.5	3 to 5	1.5
A/a	1 to 2	1	3 to 5	2

Note: d, v, and a are the displacement, velocity, and acceleration, respectively, computed for free-field conditions.

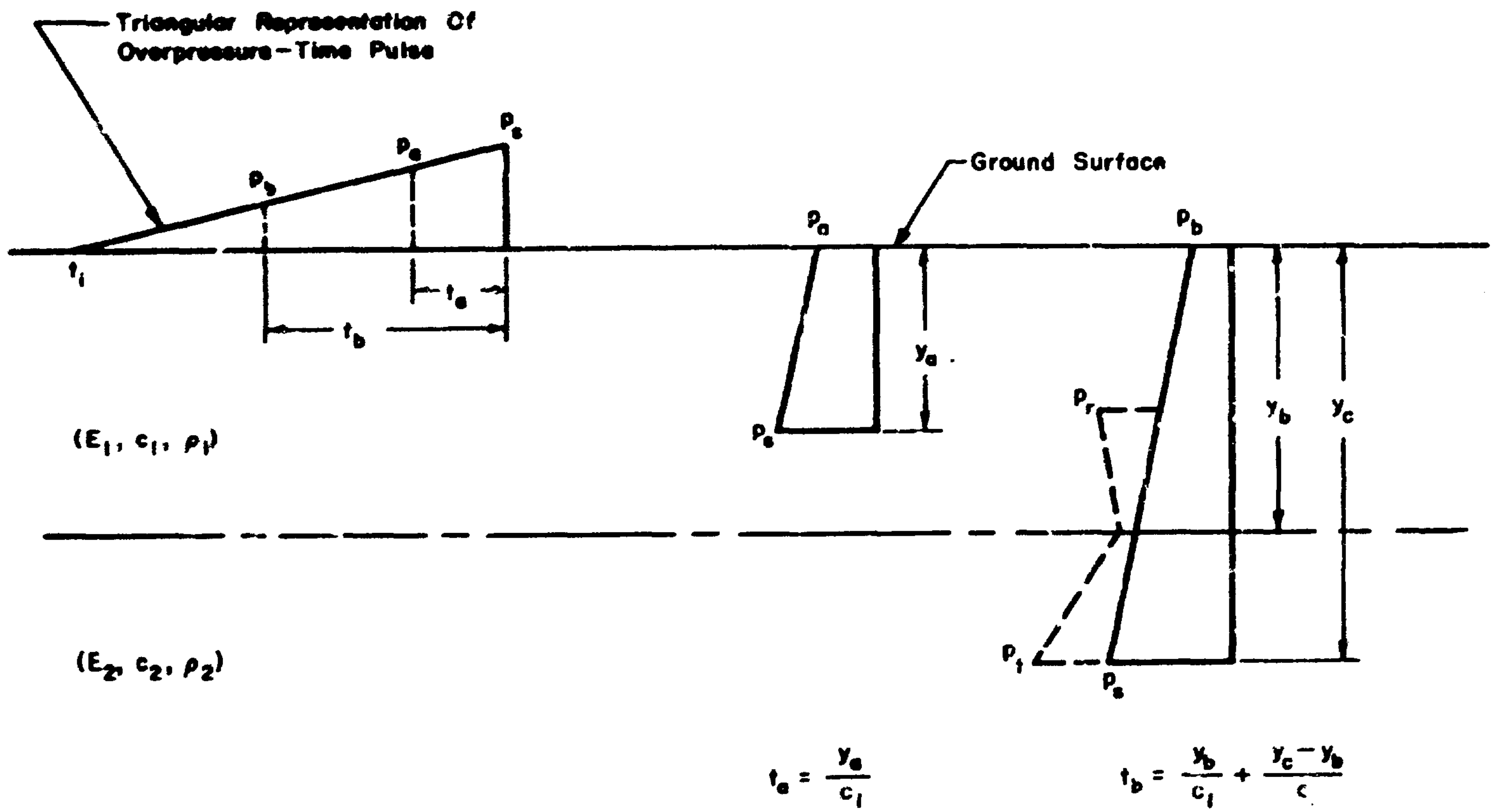


FIG. 5C-1 PASSAGE OF STRESS PULSE THROUGH SOIL

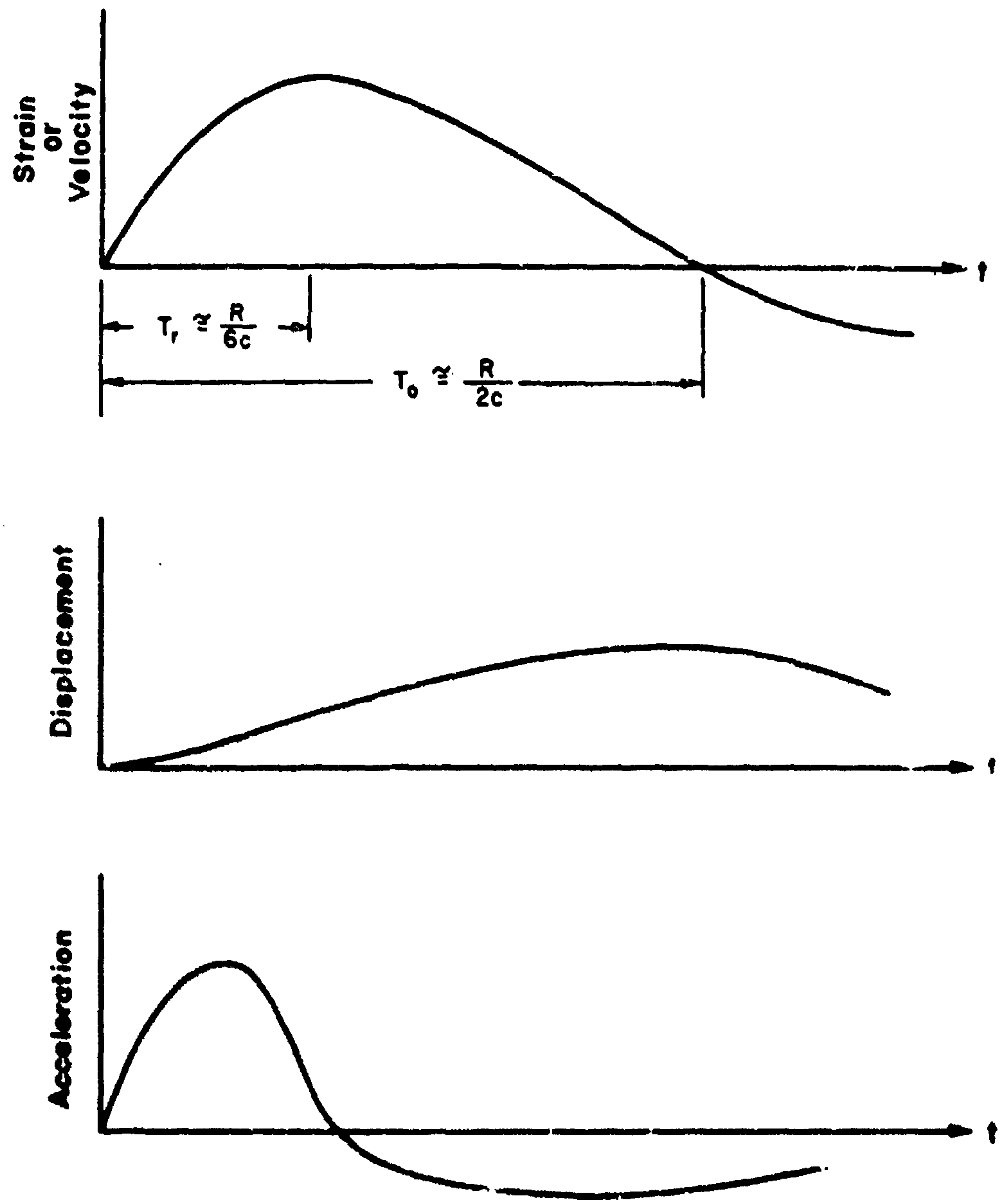
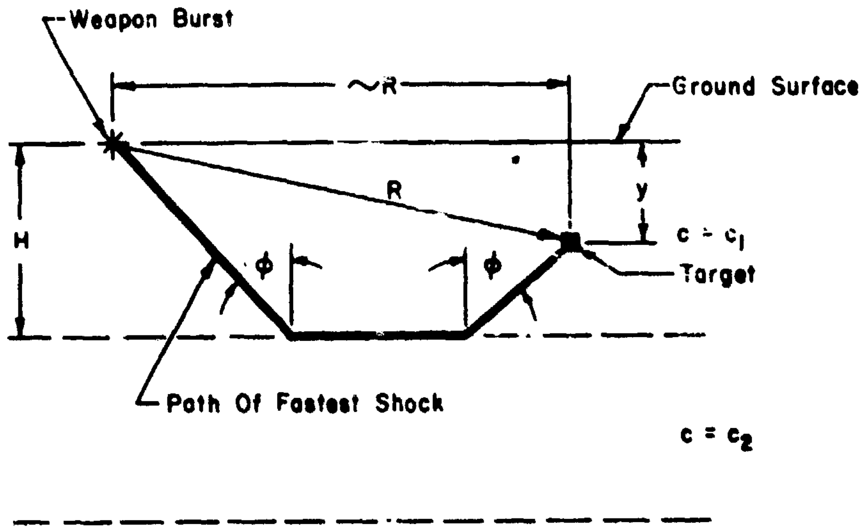
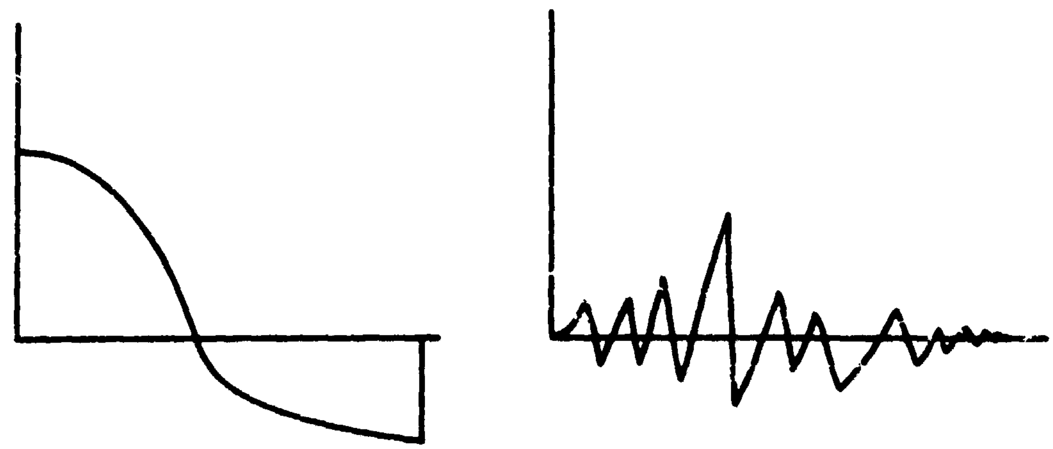


FIG. 5C-2 INITIAL PORTION OF STRAIN-TIME CURVE IN ROCK, AND CORRESPONDING DISPLACEMENT AND ACCELERATION WAVE FORMS



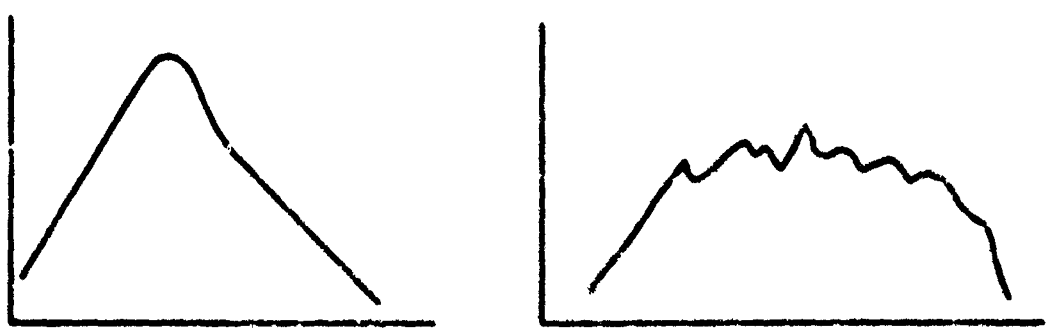
Note.
(a) R Is Much Greater Than 2H.
(b) If Target Is In Faster Layer, Use $\bar{c} = c_2$

FIG. 5C-3 DIRECT SHOCK TRANSMISSION IN A TWO-LAYERED SYSTEM



(a) Systematic (b) Random

FIG. 5C-4 COMPONENTS OF GROUND MOTION



(a) Systematic (b) Random

FIG. 5C-5 RESPONSE SPECTRA

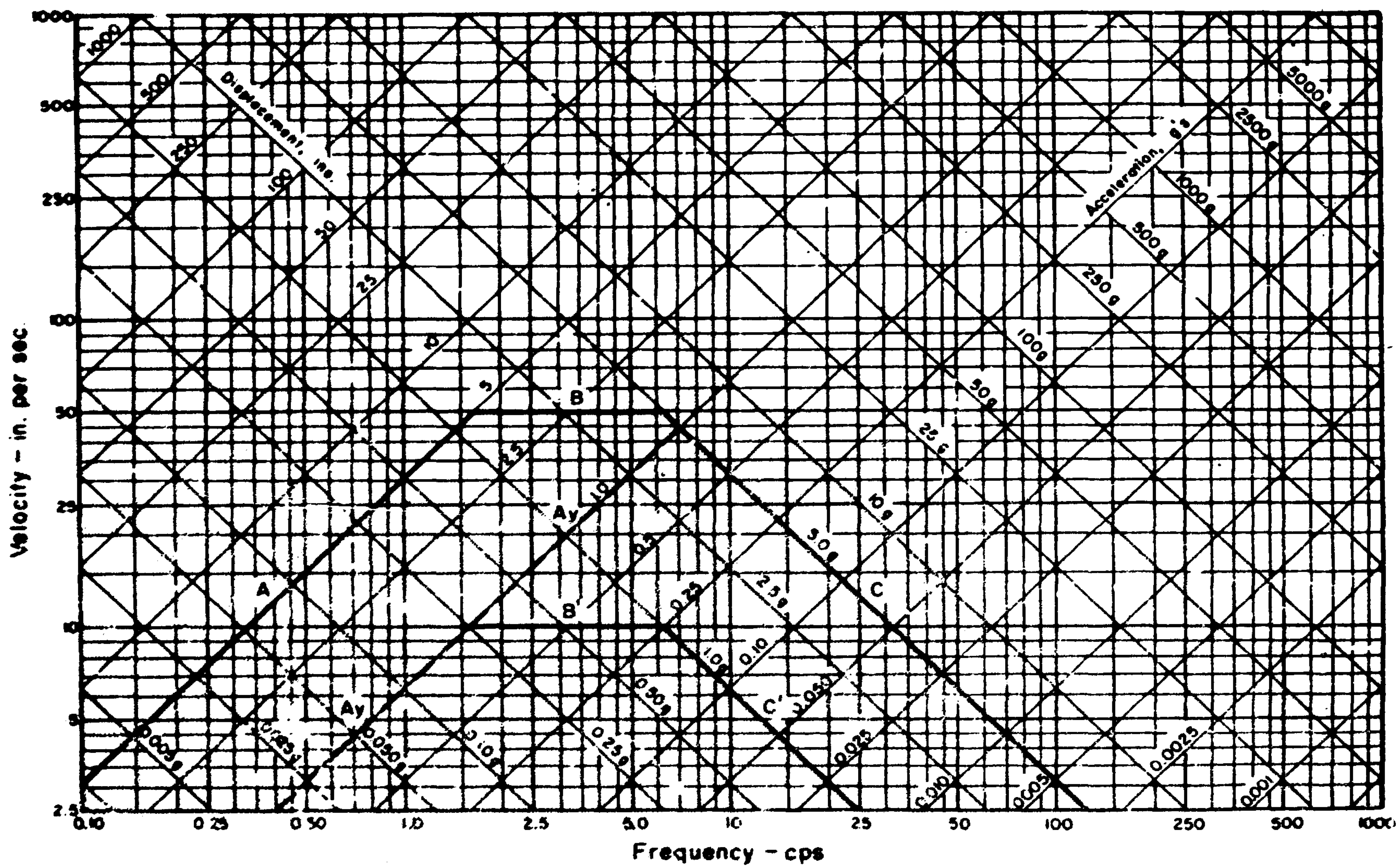


FIG. 5C-6 TYPICAL RESPONSE SPECTRUM. (Values shown have no physical significance – they are used for illustration only.)

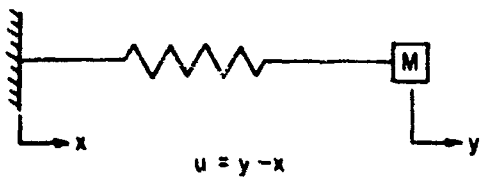


FIG. 5C-7 SIMPLE MASS-SPRING SYSTEM

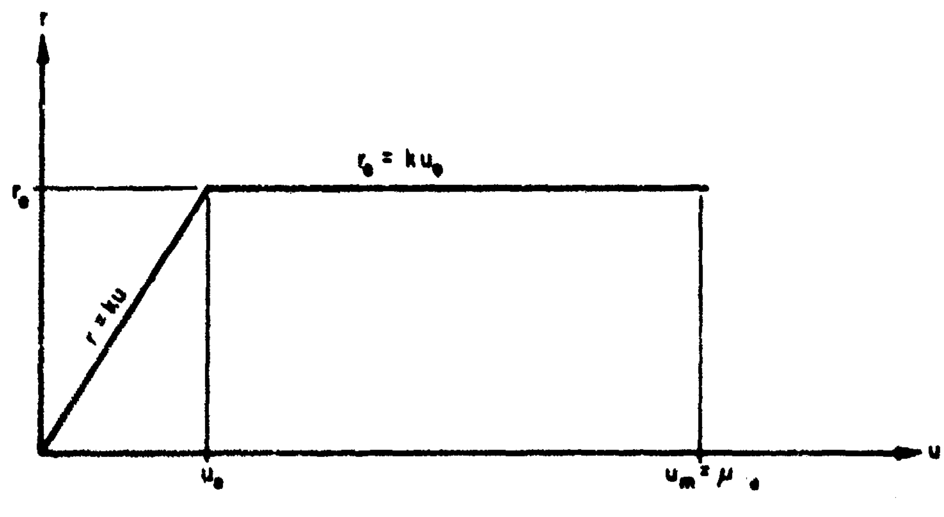


FIG. 5C-8 FORCE-DEFLECTION RELATION FOR SPRING

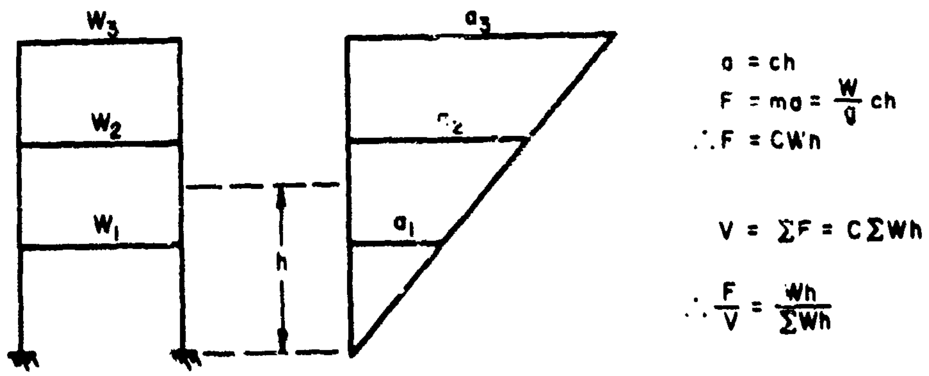


FIG. 5C-9 ACCELERATION CORRESPONDING TO DESIGN FORCE DISTRIBUTION OVER HEIGHT OF BUILDING FRAME

APPENDIX 5D. DESIGN ANALYSES FOR SPECIAL CASES

APPENDIX 5D. DESIGN ANALYSES FOR SPECIAL CASES

5D.1 GENERAL PROCEDURES FOR DYNAMIC ANALYSIS

The procedure for dynamic analysis described herein is a simplified and rapid procedure for determining the relationships between the peak dynamic force applied to a structure or structural element, the resistance of the element, the effective duration of the force, the period of vibration of the element, and the ratio of the maximum deflection of the element to the yield deflection. It is believed that the procedure described herein, although it may involve inaccuracies of the order of 20 to 25 percent in some cases, is sufficiently accurate for all practical purposes because the parameters entering into the problem are not accurately determinable. Even a much more exact analysis by procedures which involve no analytical inaccuracy could not ordinarily reduce the uncertainty below a value perhaps even greater than 25 percent because of the lack of definite knowledge in advance concerning the blast pressure for a given distance from a given energy of explosive, the duration of the blast wave, the structural parameters, and because of the general complexity of the problem.

For unusual cases, an analysis may be made using numerical methods adopting the procedures described in Refs. 5D-1 and 5D-2. These methods are generally tedious and time-consuming.

The method of analysis described herein requires a description of the loading-time curve on the structure, a knowledge of the structural resistance, the shape of the resistance-deflection curve for the structure and especially a characterization of it by a ductility parameter giving the permissible maximum deflection in relationship to the yield-point deflection of the structure, and a measure of the period of vibration in the "elastic range" of the structure. These individual items are described in the following paragraphs and finally the general dynamical relationships between these quantities are given and a chart is presented by which one can make rapid determinations of the peak loading required to produce a given dynamic deflection.

The methods described have been presented in previous publications, Refs. 5D-1 and 5D-3. They are summarized here for convenience, with some elaboration of the more complex parts of the procedure. Other general methods are described in Refs. 5D-4, 5D-5, and 5D-6.

5D.2 LOADING

Loadings on aboveground and belowground structures differ very markedly in character. On an aboveground wall making an angle of 40 degrees or less with the shock front of the blast wave, the peak force acting on the structure is equal to the reflected overpressure which is something between 2 and 8 times the peak side-on overpressure. The relation between these two is given by the following equation.

$$\frac{p_r}{p_{so}} = 2 + \frac{6p_{so}}{7P_o + p_{so}} \qquad (5D-1)$$

In this equation p_r is the peak reflected overpressure,

p_{so} is the peak side-on overpressure,

P_o is the ambient atmospheric pressure.

The reflected overpressure decays rapidly to a value corresponding to the side-on overpressure plus some proportion of the so-called drag pressure or dynamic pressure, in a time called the "clearing" time. A discussion of drag or dynamic pressures is presented in connection with Eq. (5D-5). The clearing time is a function of the distance from the stagnation point to the nearest edge. The stagnation point generally is that point furthest from all edges of the surface under consideration. In terms of this shortest distance, S, the clearing time t_c is given by the relation:

$$t_c = 3S/U \qquad (5D-2)$$

where U = velocity of shock front. After the clearing time the pressure decays with the decay in side-on overpressure and dynamic pressure to zero in an effective duration t_d. The effective duration of the side-on pressure is less than the positive phase length of the blast overpressure but the exact value does not matter greatly except in unusual cases. For overpressures below about 30 psi the duration may be taken as three-quarters the positive phase length. The effective duration of the drag pressure is roughly half as long [See Eq. (5D-6)]. For higher overpressures t_d may be considered to be given by the following equation for the duration of an equivalent triangular impulse:

$$t_i = 0.40 \text{ sec. } \left(\frac{100 \text{ psi}}{p_{so}}\right)^{0.6} \left(\frac{W}{1 \text{ MT}}\right)^{1/3} \qquad (5D-3)$$

where W equals yield of bomb in megatons.

The net force on the front wall of the structure is given in Fig. 5D-1. It can be approximated by three triangular pressure-time curves. The small upper part can sometimes be considereed as a separate impulse added to a longer duration triangle. This is true if the clearing time is relatively short in comparison with the period of vibration of the element. Methods are given in this appendix for dealing with the more general case where the clearing time is not short in comparison with the period of the structure.

The net translational force on the structure is more complex. One must consider a loading on the rear of the structure which starts at a time corresponding to the travel time over the length of the structure, and which increases in a time interval of 3S/U to a magnitude equal to the side-on overpressure minus the drag coefficient on the rear face times the dynamic pressure. A diagram of these pressures is shown in Fig. 5D-2. The difference between the pressure curves forward in Fig. 5D-1 and rearward in Fig. 5D-2

gives the net translational force on the structure. In many cases this can be approximated by a single triangle having a peak magnitude equal to the reflected pressure on the front face and a duration given by Eq. (5D-4):

$$t_1 = \frac{L + 3S}{U + 3S/t_d} \tag{5D-4}$$

In this equation L is the length of the building, and if L is less than S the numerator is to be taken as 4S. If L is greater than 3S the numerator is to be taken as 6S. In the denominator, if the second term, $3S/t_d$, is greater than U/3, the denominator is to be taken as 4U/3.

The roof slab may be considered to be exposed to the side-on overpressure only; or if desired a rise time for effective loading may be considered.

For an open structure with only beams, columns, trusses, or other members with only small areas opposing the blast, each of the members receives a small impulsive loading as the blast engulfs it and each member is then exposed to drag from the wind accompanying the blast. Because the blast is transmitted through the building in a finite time, the net translational force increases until all or nearly all the building is engulfed. However, the impulse from the diffraction around each member produces a spike on the net force diagram as the blast reaches that particular member, of a type similar to the force diagram on a rectangular building described above. Unless the building is extremely brittle and fails without plastic deformation, it is reasonably accurate to consider that the building is subjected only to drag, neglecting the individual impulse spikes, and to assume that the whole building is engulfed at one time and is subjected to a trangular force varying from a maximum equal to the drag coefficient for the items in the building times the dynamic pressure, acting on the net drag area, and varying to zero in a time equal to the effective duration of the drag force. The latter is approximately one-half the effective duration of the side-on overpressure.

The peak drag or dynamic pressure may be computed by the following equation:

$$p_d = \frac{2.5 p_{so}^2}{7P_o + p_{so}} \tag{5D-5}$$

However, in cases of high thermal effect on the ground surface the drag pressure may actually be 3 or 4 times as great as is given by this relation.

The drag coefficients to be used should take into account the shielding of elements by others placed a short distance away. However, if the distance between parallel elements is more than ten times their width the shielding is negligible. Recommended values of drag coefficients to be used are about 2 for structural shapes, about 1.5 for box-shaped elements of for flat plates, and about 0.8 for short cylinders, decreasing to 0.3 for

long cylinders. However, for high velocities or for overpressures greater than 60 psi the drag coefficients for all sections probably should be taken as 1.5.

The effective duration of the drag pressure is given by the following equation:

$$t_d = 0.18 \text{ sec.} \left(\frac{100 \text{ psi}}{p_{so}}\right)^{1.1} \left(\frac{W}{1 \text{ MT}}\right)^{1/3} \tag{5D-6}$$

For underground structures data indicate that probably no significant reflections occur when the pressure wave in the soil engulfs the structure. However, the pressure loading on the structure differs from the free-field stresses because of arching effects. The design loads for this class of structures may be determined in accordance with the procedures given in SECTION 5.3 concerning loadings on tunnel type structures.

In general, the loading is then described as a pressure-time diagram where the peak force on the structure is defined, and the variation of force with time assumes a shape which can usually be approximated by several straight lines forming a polygon usually concave upward, as in Fig. 5D-1. In most cases, only two straight-line parts are necessary, with the provision that it is desirable to maintain the actual value of the total peak overpressure and the area under the curve should be preserved roughly to the point where the first triangle ends, and the second triangle should include all of the remainder of the impulsive force transmitted.

5D.3 STRUCTURAL RESISTANCE

The relation between load and deformation or between resistance and deflection of a structure or structural element may take any of a number of forms. It is convenient to define the structural resistance R in the same terms and in the same units as the external loading p. Then the relation between R and the deflection x may be taken as in Fig. 5D-3 where there is an initial elastic part and an inelastic part with roughly a uniform resistance after yielding. The curve shown can usually be approximated by an elasto-plastic curve consisting of an initial straight line not necessarily the same as the actual initial straight-line part, and a horizontal second part which preserves the total area under the given curve and also the area at or near the yield value for the approximating curve. The elasto-plastic curve is characterized by three parameters; namely, the yield resistance r_u, the yield displacement x_y, and the maximum displacement x_m which is equal to the quantity μx_y, where μ is the ductility factor for the structure. It should be noted that the yield deflection is not the actual value at which yielding begins, but a sort of effective or equivalent value, if the true load deflection relationship is not an elasto-plastic one.

Values of yield resistances for various structural types are summarized in APPENDIX 5B.

5D.4 STRUCTURAL DUCTILITY

The ductility factor for various structures ranges from slightly greater than one for brittle structures to values of the order of 20 to 30 for very ductile structures. In some cases it can be even higher. However, unless the load duration is extremely short, as it would be in the case of high explosive bombs, it makes very little difference what the ductility factor is so long as it is greater than something in the range of about 3 to 5. In any case, if one can make an estimate of the ductility which one wishes to mobilize, he can use the actual figures in the relationships in the charts given in this section.

It is recommended that a value of $\mu = 1.3$ be taken for relatively brittle structures because there is practically no structure which does not have some inelastic deformation even up to the point of so-called yielding. For moderately brittle structures μ can be taken in the range from about 2 to 3, for the majority of ductile structures about 4 to 8, and for quite ductile structures μ can be taken in the range from 10 to 20. In general for reinforced concrete structures or for steel structures, the ductility factor for members in bending is less for deep members than for shallow members, and in reinforced concrete in particular it is less for heavily reinforced members than for lightly reinforced members. The ductility factor for compression members should be taken in the brittle range, namely about 1.3.

5D.5 PERIOD OF VIBRATION

We are concerned with the vibration of a structure in a mode most nearly like that which corresponds to the shape in which it fails. For uniform loading corresponding to blast loading this is generally about the same as the fundamental mode of vibration of a complex structure. In those instances where it is not, one can make an estimate of the period of vibration in the mode corresponding to the configuration as it approaches failure, by using methods similar to Rayleigh's method.

A simple procedure which one can adopt to determine the first mode is to assume the deflected shape of the structure as it looks when approaching failure, take inertia forces proportional to the masses at the various points of deflection of the structure multiplied by the square of the circular frequency of vibration and by the deflection in the assumed mode shape, and apply these inertia forces to the structure assuming it to have no mass. The deflection of the structure is then computed, and the deflected shape will in general be different from the shape assumed. The value of the circular frequency which makes the assumed shape most nearly equal to the derived shape is that which should be used in computing the natural period of vibration. One obtains from this procedure the square of the circular frequency, from which the circular frequency can be derived, and the period is obtained from the circular frequency ω by the relationship

$$T = 2\pi/\omega \qquad (5D\text{-}7)$$

in which T is the period of vibration. The same procedure may be used to fine the next higher modes, if the earlier mode components are subtracted out. A more general approach for multi-degree-of-freedom systems is given in SECTION 5C.8(j).

In general the calculation for the natural period will correspond to the initial straight-line part of the actual load deflection curve for the structure. In order to be accurate we should use the period corresponding to the stiffness of the equivalent elasto-plastic structure as defined by the approximating initial straight-line part in Fig. 5D-3. The effective period to be used in further calculations is obtained from the period for the initial elastic stage by dividing the latter by the square root of the ratio of the slope of the equivalent elastic resistance to the slope of the initial elastic resistance. In other words, the equivalent elastic resistance has a slope which is smaller than that for the initial elastic resistance, or the structure has a smaller effective modulus than in the initial elastic state. When we divide the initial elastic period by the square root of a ratio less than one we obtain a larger number which indicates that the effective period for the effective elasto-plastic resistance curve is longer than the period corresponding to the initial slope of the resistance curve.

Estimates of the equivalent period of vibration for a number of different types of structures are given in APPENDIX 5B as a function of the dimensions and masses of the structural elements.

5D.6 GENERAL DYNAMIC RELATIONS

With the simplifications outlined in the preceding sections, the structure is essentially reduced to an equivalent simple one-degree-of-freedom system. Solutions for such systems have been obtained for a number of types of loading for various kinds of resistance curves. A simple chart which gives the relationships among the various parameters is shown in Fig. 5D-5. This gives the relations among the following four quantities:

x_m/x_y or ductility factor, which is shown on the scale on the left-hand side of the figure.

t_d/T or the ratio of the duration of a triangular load pulse to the effective period of the structure, shown as the abscissa.

p_m/q_y equals the ratio of the peak dynamic force applied to the structure to the effective yield point resistance of the structure or structural element. Lines of equal values for this ratio are shown by the lines which slope generally diagonally up and to the right of the figure.

t_m/T or the ratio of the time at which maximum deflection is reached to the natural effective period of vibration. These are shown by the dotted lines sloping generally down and to the right on the figure.

In general, for design purposes the value of the ductility factor and the relative duration can be estimated, and the value of the ratio of the

peak dynamic force to the yield point resistance is determined from the chart. Then if the peak dynamic force is given, the yield-point resistance can be determined or if the yield-point resistance is given the peak dynamic force can be determined. In either case, the duration or the period of vibration may not be known until the other quantities are determined, but by a recalculation and a re-use of the chart, one can finally come up with the necessary result.

It may be of interest to determine the time at which yielding begins. This is given by

$$t_y/T = 0.23\sqrt{q_y/p_m} \tag{5D-8}$$

In this equation t_y is the time at which yielding starts. Equation (5D-8) is valid if the effective duration of the applied pressure is greater than t_y.

For more complex loading curves, it is still possible to use the chart in Fig. 5D-5 with a reasonably accurate degree of approximation by the following procedure.

Let us assume we have a loading curve of the form given in Fig. 5D-4 where, for convenience, only three separate triangular elements are considered in the loading curve. More or fewer can be treated in exactly the same way. For each elementary triangle we have a partial loading p_1, p_2, or p_3. We can designate the general expression as p_n. For each triangle we also have the duration, with a corresponding subscript, t_1, t_2, or t_3, or for the general case t_n. It is assumed that the value of the effective period of vibration of the structure, T, and the ductility factor, μ, are known. It is required to determine the required yield resistance q_y. We proceed as follows:

For any component of loading, having a duration t_n, use the chart to determine for the given period of vibration and the ductility factor desired the ratio of p_n to q_y. Let this quantity be denoted by the symbol F_n. For the loading diagram in Fig. 5D-4, we will have determined values of F_1, F_2, and F_3. These will be determined for the same ductility factor and same period of vibration, of course. We now apply the general approximate relationship:

$$\Sigma \frac{p_n/q_y}{F_n} = 1 \tag{5D-9}$$

In the case of three component loadings this reduces to

$$\frac{p_1/q_y}{F_1} + \frac{p_2/q_y}{F_2} + \frac{p_3/q_y}{F_3} = 1$$

If we multiply both sides of the latter equation by q_y we obtain the result:

$$q_y = \frac{p_1}{F_1} + \frac{p_2}{F_2} + \frac{p_3}{F_3} \tag{5D-10}$$

In the general cas(...)n given by

$$-(p_n/F_n) \qquad (5D\text{-}11)$$

As an illustration of the procedure let us consider a situation where we have a loading diagram in which we have only two components, where p_1 is 80 psi and p_2 is 20 psi. Let us take t_1 as 0.10 sec. and t_2 as 1.00 sec., and the period of vibration T as 0.20 sec. Let us assume a ductility factor of 3. Then since the first duration is only half the period of vibration, we find from the chart in Fig. 5D-5 a value of F_1 of 1.75; and for F_2 a value of 0.90, since the ratio of this duration to the period of vibration is 5.0. Then by use of Eq. (5D-11) we find the following result:

$$q_y = \frac{80}{1.75} + \frac{20}{0.90} = 45.8 + 22.2 = 68 \text{ psi}$$

The required yield-point resistance is 68 psi. It will be noted that if the entire initial overpressure of 100 psi had been applied over a period of time of 0.10 sec., the yield-point resistance needed would have been 57 psi instead of 68 psi. On the other hand, if the entire 100 psi had an effective duration of 1 sec., the yield-point resistance needed would have been 111 psi. This is considerably greater than the value of 68 psi required for the actual loading.

If we had been designing the structure, unless we were able to make a fairly good guess initially, we would have had to recompute the period of vibration corresponding to the new design value of the yield point. We would then repeat the calculations until we find something that checks our assumed period of vibration.

5D.7 REBOUND

When any structure is loaded, and it reaches a maximum deflection, it has energy stored in it and tends to deflect backward, or in the opposite direction. This tendency exists even in the case when there is still some forward loading action on the structure at the time it reaches its maximum deflection. In general, the rebound is elastic although in the case of a reinforced concrete structure, if the rebound is very large and sufficient rebound steel is not provided, there may be an inelastic part of the rebound curve. It is conservative to design on the basis of an elastic rebound situation. If this is done, one can determine the required rebound resistance, in terms of the design yield-point resistance for the forward direction. A chart giving the ratio of the rebound resistance r, in relationship to the yield resistance q_y, is given in Fig. 5D-6. It should be noted that for short duration loadings relative to the period of vibration of the structure, the ratio of rebound resistance to yield resistance is -1.0 which means that there is a full 100 percent rebound. For long duration loadings, the ratio drops.

Consider for example the situation where the ratio of loading to period is 5.0 and the ductility factor is 3.0. From the curve, by interpolating between the lines, one finds a required rebound resistance of -0.35 times the

yield resistance. This means that in this case a rebound reaction of 35 percent of the yield value is necessary and reinforcement in a concrete member of the order of 35 percent of the positive reinforcement would be required for rebound.

The rebound values given in the chart in Fig. 5D-6 may be somewhat conservative because of the neglect of loss of energy due to damping and also because of the fact that the curves are computed for the maximum rebound reaction assuming the most unfavorable duration in those instances where a slight variation in duration would make a difference in the required rebound force. The assumption of no damping is perhaps the most serious one. The maximum rebound occurs very late in the loading curve or after the loading has been applied, in the free vibration period following it. Consequently, there may be many oscillations with consequent loss in vibratory energy, before the maximum rebound is developed. However, in the absence of more definitive information, it is recommended that provision be made for the rebound forces shown in Fig. 5D-6. Of course, the dead load and superimposed masses acting on a structure can reduce the required rebound resistance since these forces may act only in the forward direction. Rebound resistance must be provided in the member itself as well as in hold-down lugs or other appurtenances at the reactions.

5D.8 REFERENCES

5D-1. Newmark, N. M., "Analysis and Design of Structures to Resist Atomic Blast", Bulletin Virginia Polytechnic Institute Eng. Exp. Sta. No. 106, Part II, pp. 49-77, January 1956.

5D-2. Newmark, N. M., "A Method of Computation for Structural Dynamics", Journal of the Engineering Mechanics Division American Society of Civil Engineers, July 1959.

5D-3. Newmark, N. M., "An Engineering Approach to Blast Resistent Design", Transactions American Society of Civil Engineers, Vol. 121, 1956, pp. 45-64.

5D-4. "Design of Structures to Resist the Effects of Atomic Weapons", Manual Corps of Engineers, U. S. Army, EM 1110-345-414, March 1957.

5D-5. "Design of Underground Installations in Rock", Manual Corps of Engineers, U. S. Army, EM 1110-345-431. (In Preparation)

5D-6. Merritt, J. L. and Newmark, N. M., "Design of Underground Structures to Resist Nuclear Blast", Vol. 2, Final Report, Contract DA-49-129-eng-312, University of Illinois and Office Chief of Engineers, April 1958.

5D-7. Newmark, N. M. and Hall, W. J., "Preliminary Design Methods for Underground Protective Structures," Report prepared under Contract AF29(601)-1171 for the Air Force Special Weapons Center, Kirtland Air Force Base, New Mexico, December 1959, AFSWC-TR-60-5 (Secret).

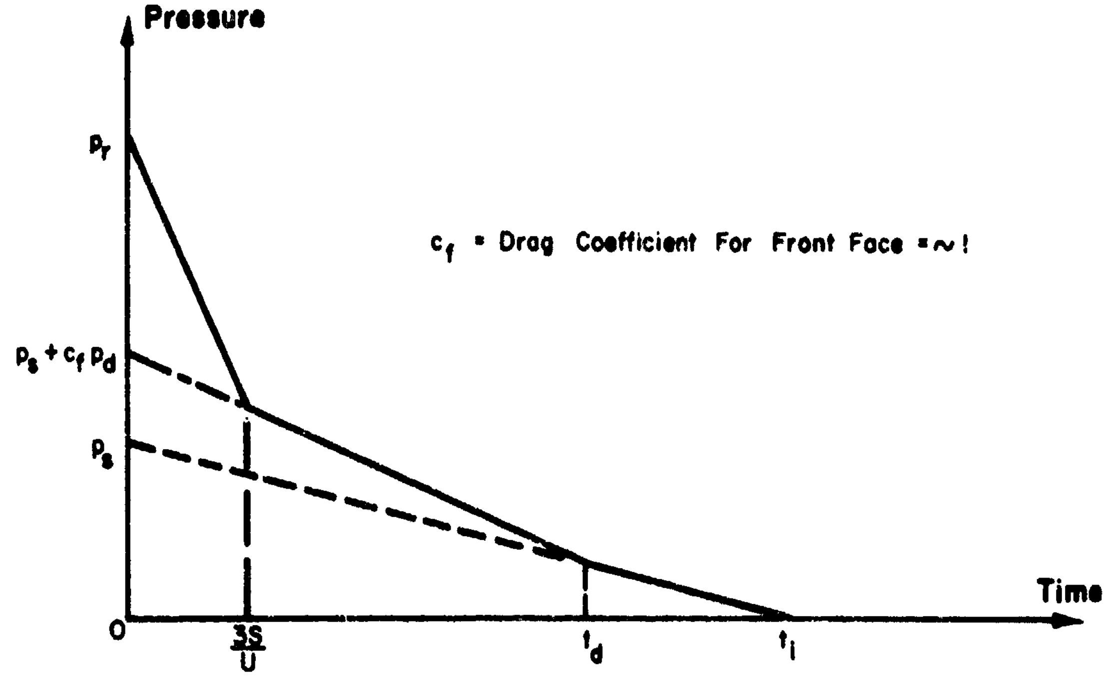


FIG. 5D-1 NET PRESSURES ON FRONT OF RECTANGULAR BUILDING

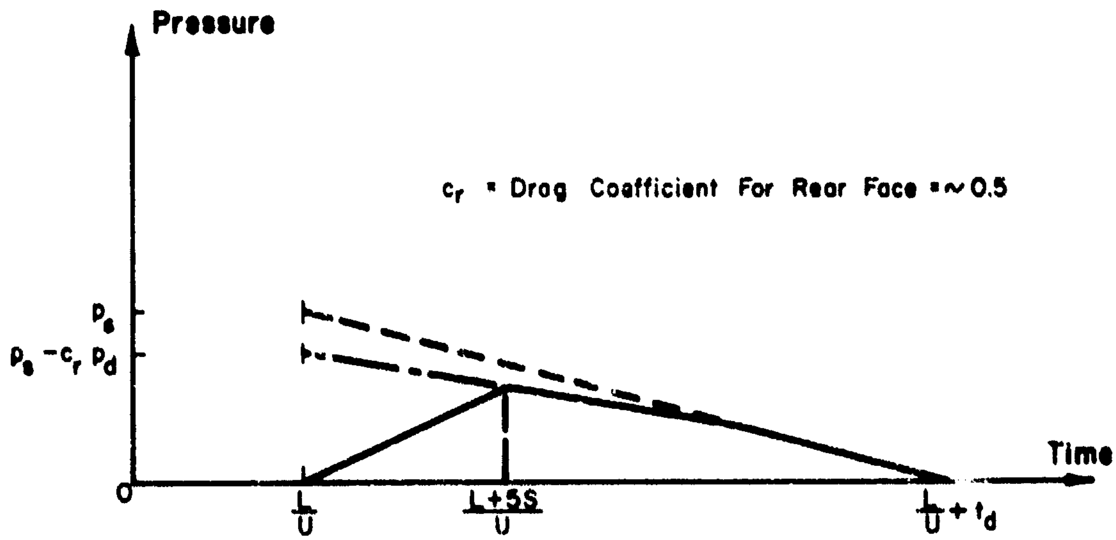


FIG. 5D-2 NET PRESSURES ON REAR OF RECTANGULAR BUILDING

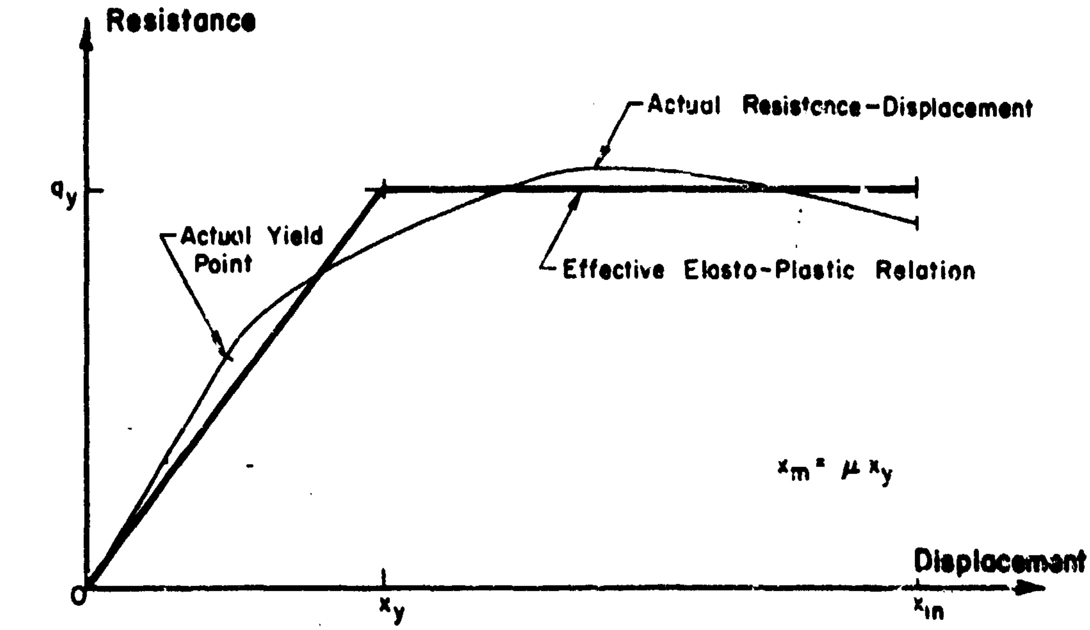


FIG. 5D-3 RESISTANCE—DISPLACEMENT RELATIONSHIP

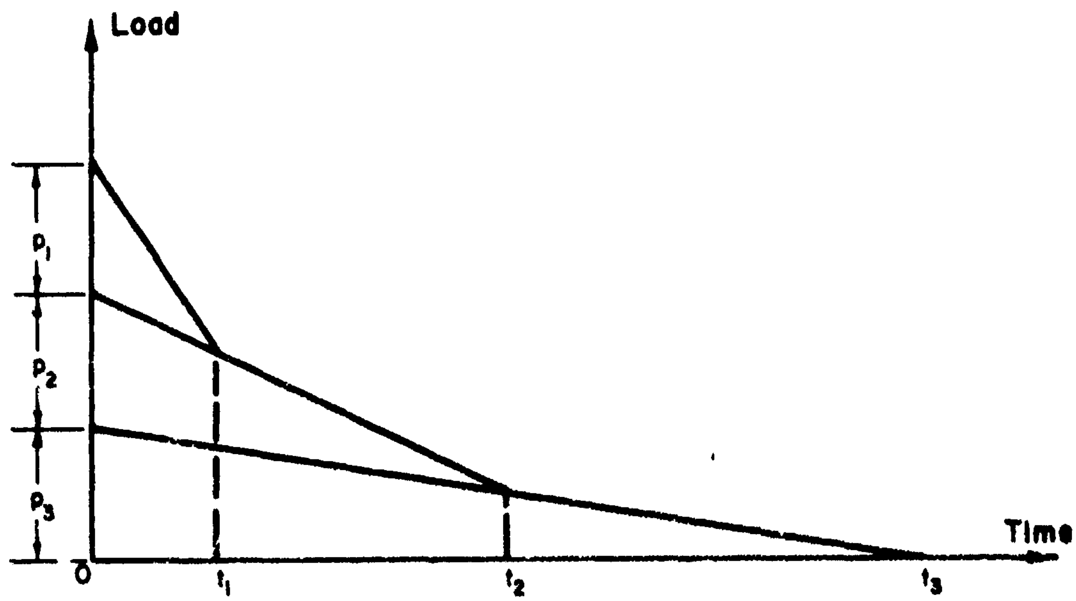


FIG. 5D-4 TYPICAL LOAD-TIME RELATION CONSIDERED

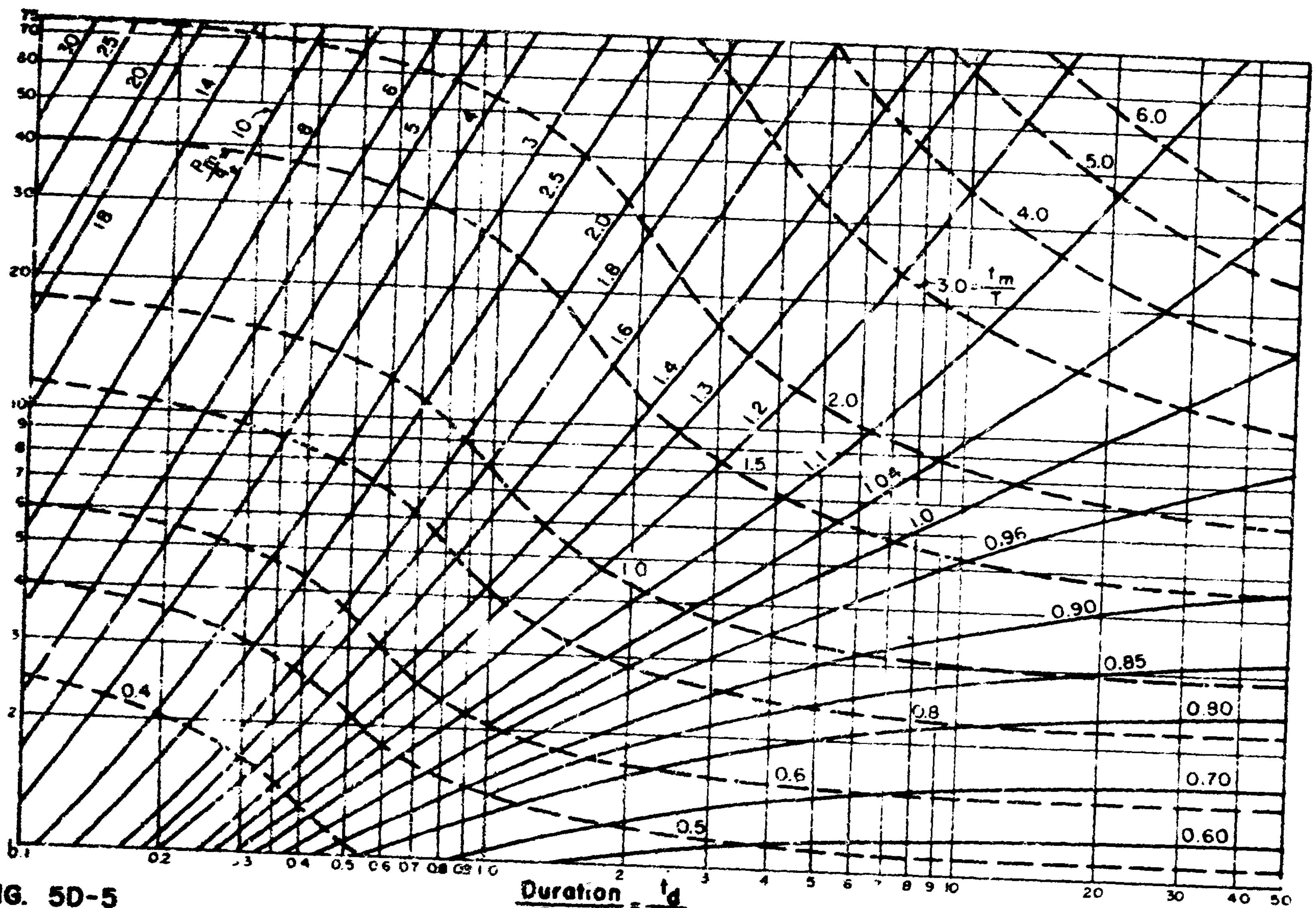


FIG. 5D-5
DESIGN CHART FOR INITIAL PEAK TRIANGULAR FORCE PULSE ON ELASTO-PLASTIC SYSTEM

NEWMARK, HANSEN & ASSOCIATES

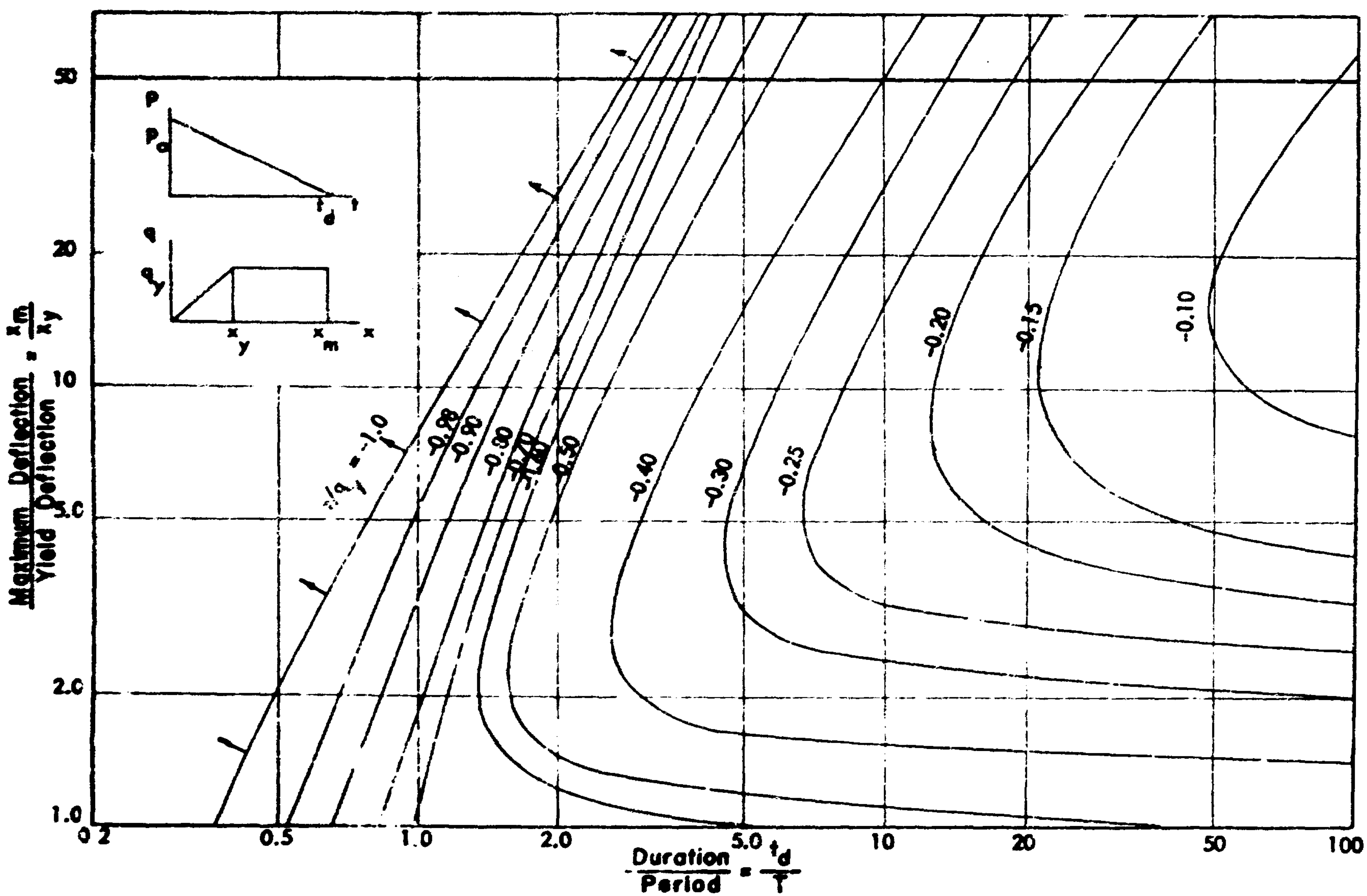


FIG. 5D-6 DESIGN CHART FOR ELASTIC REBOUND

APPENDIX 5E. SHIELDING CRITERIA FOR GAMMA RADIATION FROM FALLOUT

APPENDIX 5E. SHIELDING CRITERIA FOR GAMMA RADIATION FROM FALLOUT

5E.1 INTRODUCTION

Although the primary emphasis in this Review Guide has been given to the resistance of the immediate effects of blast and radiation, there is nevertheless a very real hazard to personnel from the gamma radiation caused by fallout. Fallout can occur over wide areas and can last for several days. A locality which escapes the immediate effects of blast and radiation may yet receive a lethal rain of fallout particles. It is conceivable, therefore, that in the preparation or review of Protective Construction planning one may wish to evaluate a structure in its effectiveness for fallout shielding.

Several procedures for shelter evaluation are available: CBR EM-1110; TP-PL-8; and the one discussed herein, taken from Ref. 17, which will be referred to as the OCDM Manual. The OCDM Manual is selected for discussion because it is the recent revision of the material presented in the original Protective Construction Review Guide; and because it is a relatively simple but still sound approach.

The reader may prefer one of the other manuals from experience, or he may wish to compare results from more than one. There are significant differences among these sources concerning the energy of fallout and corresponding shielding effectiveness of various construction materials. No attempt is made herein to resolve these differences nor to judge the relative accuracy or correctness of any given procedure. In view of the major variables involved in the assumed fission yields, ground zero, and wind and time factors in a fallout shelter analysis, and considering the fact that a change in sheltering techniques equivalent to only two inches of concrete changes the attenuation by a factor of approximately two, the quantitative differences in the results of the various methods listed above are not considered critical for the purposes of this document.

5E.2 CONSIDERATION OF FALLOUT

From SECTION 2 one can predict the total fallout gamma dosage to be expected for a situation with a specific set of weapon and structure parameters. From SECTION 4 the dosage tolerance for personnel can be determined. Comparison of the expected dosage and the maximum dosage which can be tolerated gives a measure of the sheltering or shielding required.

In the case of completely buried, underground facilities it is usually sufficient to use a curve such as Fig. 5-9, which gives shielding properties of various materials to residual gamma radiation. This curve will enable one to determine the necessary thickness of material to achieve the desired attenuation, or conversely to find the attenuation which will be provided by a given physical barrier.

In the case of structures which are partly or completely aboveground, or in the case of exposed entranceways and apertures connected with buried structures, the procedures for shelter evaluation are useful.

5E.3 OCDM MANUAL PROCEDURE

The procedure given in the OCDM Manual is suited for evaluating existing or proposed structures of more or less conventional type. The procedure calculates and compounds reduction factors which are a measure of the ratio of the radiation at a shielded detector to that of the unshielded contaminated plane. Taking into account the effects of barriers and geometry, the procedure leads to an over-all reduction factor applicable to a given location in the interior of the structure.

A number of assumptions are made. The fallout is assumed to be uniformly distributed over exposed surfaces according to their horizontal projections. No fallout is assumed to remain on vertical surfaces. The energy spectrum of the gamma radiation is taken as that from fission products at one hour following the weapon burst. The attenuation produced by a given barrier is assumed to be a function only of its mass thickness. For ground floor areas the detector position is assumed to be three feet above ground level at the geometric center of the structure. For basement areas the detector position is assumed to be five feet below the level of the ground floor at the geometrical center of the structure. The radiations reaching the detector are grouped into that caused by fallout on the roof (Roof Contribution) and that caused by fallout on the ground surface (Ground Contribution). Correction factors are provided to correct for several effects in a qualitative way.

Because of space limitations the OCDM procedure will be presented without detailed explanation of all features and refinements. For more thorough information the reader is referred to the OCDM Manual itself.

5E.4 PRESENTATION OF CHARTS AND TABLES

The following tables and charts are taken from the OCDM Manual.

5E.4.1 Mass Thickness. Table 5E.1 gives the mass thicknesses of common construction materials in pounds per square foot of surface area for given thicknesses. It has been assumed that the weights given for various types of floor, roof, and wall construction in standard engineering tables are equivalent to the mass thickness of the construction.

5E.4.2 Charts. Figures 5E-1 through 5E-6 enable one to calculate reduction factors for most situations. Figure 5E-1 gives the barrier shielding effects as shown. Figure 5E-2 gives the reduction factors for combined shielding effe[illegible] to be applied to the roof contribution. Figures 5E-3 and 5E-4 give the abov[illegible]und detectors and belowground detectors the reduction factors for combined shielding effects to be applied to the ground contribution. Figure 5E-5 [illegible]rrects for height of detector and gives corrections to apply to the ground

contribution reduction factor. These corrections permit evaluation of the upper stories of buildings, the correction factor being applied to the value obtained for ground level. Figure 5E-6 gives reduction factors for apertures in the wall at the ground level. The use of these charts is facilitated by the tabular forms which are given in Para. 5E.5.

5E.4.3 Correction Factors. Table 5E-2, which is from the OCDM Manual, gives correction factors for the mutual shielding effects of adjacent structures. Two other tables of corrections appearing in the OCDM Manual, those for Apertures in Upper Stories and those for Skyshine Effects on Ground Contribution, are not included here since they are not used in the condensed analysis.

5E.5 TABULAR FORMS FOR EVALUATING SHELTERS

The tabular forms to follow are for a condensed shielding analysis. They are adapted from the OCDM Manual. If the interior walls are heavy, i.e., of the order of 60 psf., it is desirable to use the more complete tabular analysis form appearing in the OCDM Manual.

a) Condensed Shielding Analysis for Aboveground Areas

1) Roof Area, A_r . . . . ________
2) Distance, Roof to Detector, Z . . . . ________
3) Total Overhead Mass Thickness, X_o . . . . . ________
4) Wall Mass Thickness, X_w . . . . ________
5) % Apertures . . . . ________
6) Roof Contribution: 1), 2), and 3) Fig. 5E-2 . . . . ________
7) 1) and 4), Fig. 5E-3 . . . . ________
8) 1) and 0 psf, Fig. 5E-3 . . . . ________
9) 7) x (100% - % Apertures) . . . . ________
10) 8) x 5) . . . . ________
11) 9) + 10) . . . . ________
12) Ground Contribution, 11) x Table 5E-2 . . . ________
13) Reduction Factor, 6) + 12) . . . . ________
14) Protection Factor, reciprocal of 13) . . . . ________

b) <u>Condensed Shielding Analysis for Belowground Areas</u>

1) through 5) same as for aboveground areas. . . ______

6) Ceiling Mass Thickness, X_o' . . . . ______

7) Basement Wall Mass Thickness, X_b. . . . . . ______

8) % Exposure. . . . . ______

9) Roof Contribution, 1), 2) and 3) and Fig. 5E-2 . . . . ______

10) 4) x (100% - % Apertures) . . . . ______

11) 1) and 10) and Fig. 5E-4. . . . . ______

12) 6), Case 3, Fig. 5E-1 . . . . ______

13) 11) x 12) . . . . ______

14) 1) and 7) and Fig. 5E-3 . . . . ______

15) 14) x 8) . . . . ______

16) 13) + 15) . . . . ______

17) Ground Contribution, 16) x Table 5E-2 . . . ______

18) Reduction Factor, 9) + 17). . . . . ______

19) Protection Factor, reciprocal of 18). . . . ______

TABLE 5E-1

MASS THICKNESSES

Item	Nominal Thickness or Width of Unit	Mass Thickness Pounds Per Square Foot
Asbestos Board	3/16"	2
Asbestos, Corrugated	---	4
Asbestos Shingles	5/32"	2
Asphalt Roofing (3 ply)	---	1
Asphalt Roofing (4 ply + gravel)	---	6
Asphalt Roofing (5 ply + gravel)	---	7
Asphalt Shingles	---	2
Clay Brick	per inch	8-10
Clay Tile Shingles	---	10-20
Clay Tile, Structural	8"	42
Clay Tile, Structural	12"	58
Clay Tile, Partition	4"	18
Clay Tile, Partition	6"	28
Clay Tile, Partition	8"	34
Clay Tile, Partition	10"	40
Clay Tile, Facing	2"	15
Clay Tile, Facing	4"	25
Clay Tile, Facing	6"	38
Concrete		
Light Weight	per inch	6-8
Haydite	per inch	8
Cinder	per inch	9
Slag	per inch	10-11
Stone or Gravel (Standard Weight)	per inch	12-12 1/2
Reinforced	per inch	12 1/2
Concrete Hollow Block, Stone or Gravel (Standard Weight)	4"	30
	6"	42
	8"	55
	12"	85

Item	Nominal Thickness or Width of Unit	Mass Thickness Pounds Per Square Foot
Concrete Hollow Block, Cinder	4"	22
	6"	30
	8"	39
	12"	61
Concrete Hollow Block, Lightweight	4"	20
	6"	28
	8"	38
	12"	55
Fiber Board or Sheathing	1/2"	1
Glass	1/4"	3 1/2
Gypsum Block	2"	8-10
Gypsum Block	4"	10-12
Bypsum Board or Sheathing	4"	2
Plaster Applied Directly or on lath	1/2 - 3/4"	5-6
Plaster, Solid	per inch	8-10
Plywood Sheathing	3/8"	1
Slate	3/16"	7
Soil, Clay	per inch	6-8
Soil, Loam	per inch	7-9
Soil, Sand and Gravel	per inch	8-10
Steel	per inch	41
Steel, Corrugated Sheet	20 Ga	2
Steel Panels	18 Ga	3
Stone Masonry	per inch	10-14
Stucco	3/4"	6-9
Terra Cotta	1"	5
Wood Sheathing	1"	2 1/2
Wood Shingles	---	2 1/2
Wood Siding	per inch	1-2 1/2

TABLE 5E-2

MUTUAL SHIELDING CORRECTION TO GROUND CONTRIBUTION

Intervening Dist. to Adjacent Structure	Correction Factor
0 feet	0.00
10	0.08
20	0.10
50	0.20
100	0.40
200	0.60
500	0.80
1000	0.90
Infinite	1.00

Note:

In the case of urban buildings (those in areas of predominantly multistory commercial buildings) the correction factors tabulated above should be adjusted as follows:

Streets on all sides.......... Above values x 1.00

Streets on 3 sides............ Above values x 0.75

Streets on 2 sides............ Above values x 0.50

Streets on 1 side............. Above values x 0.25

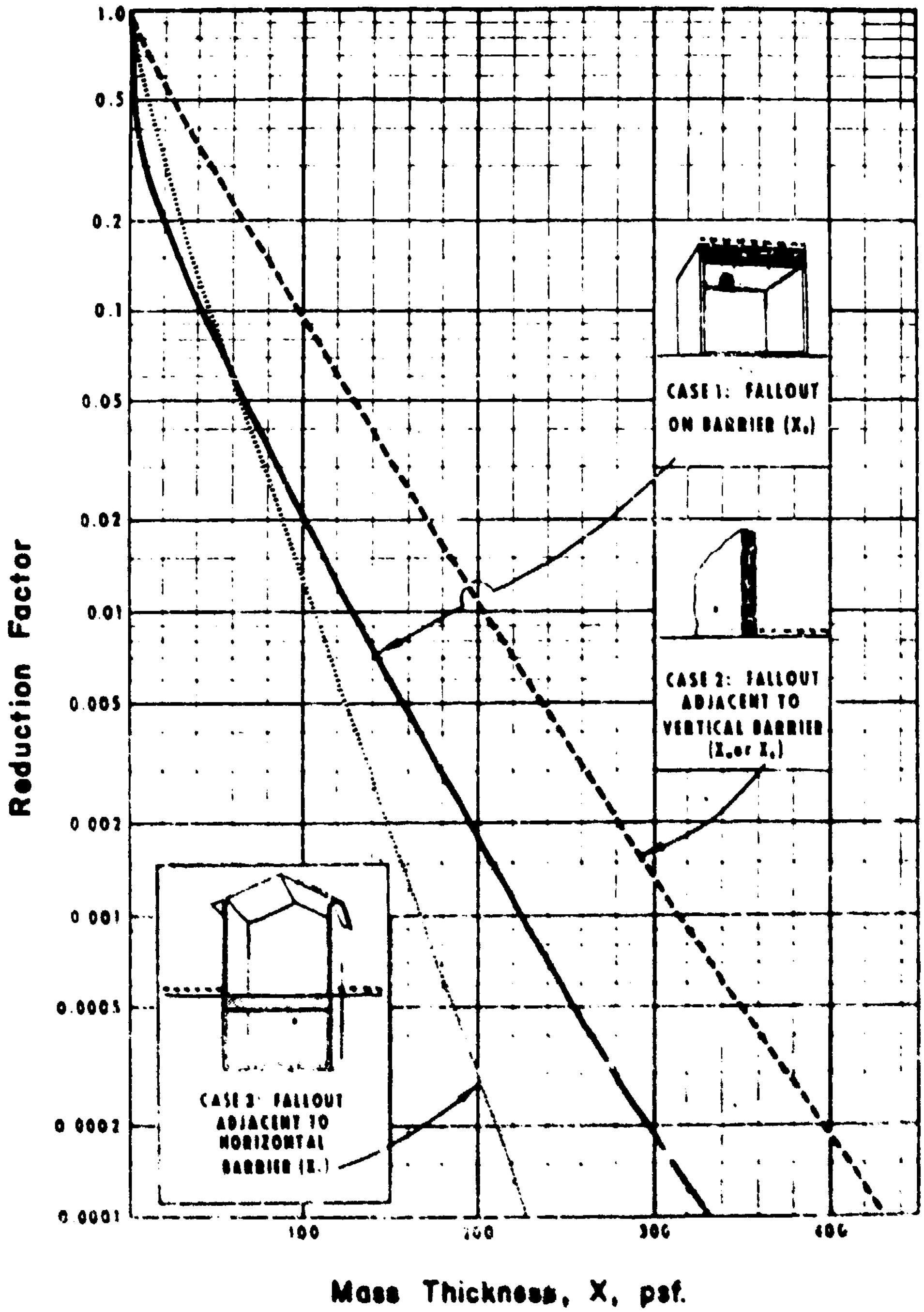
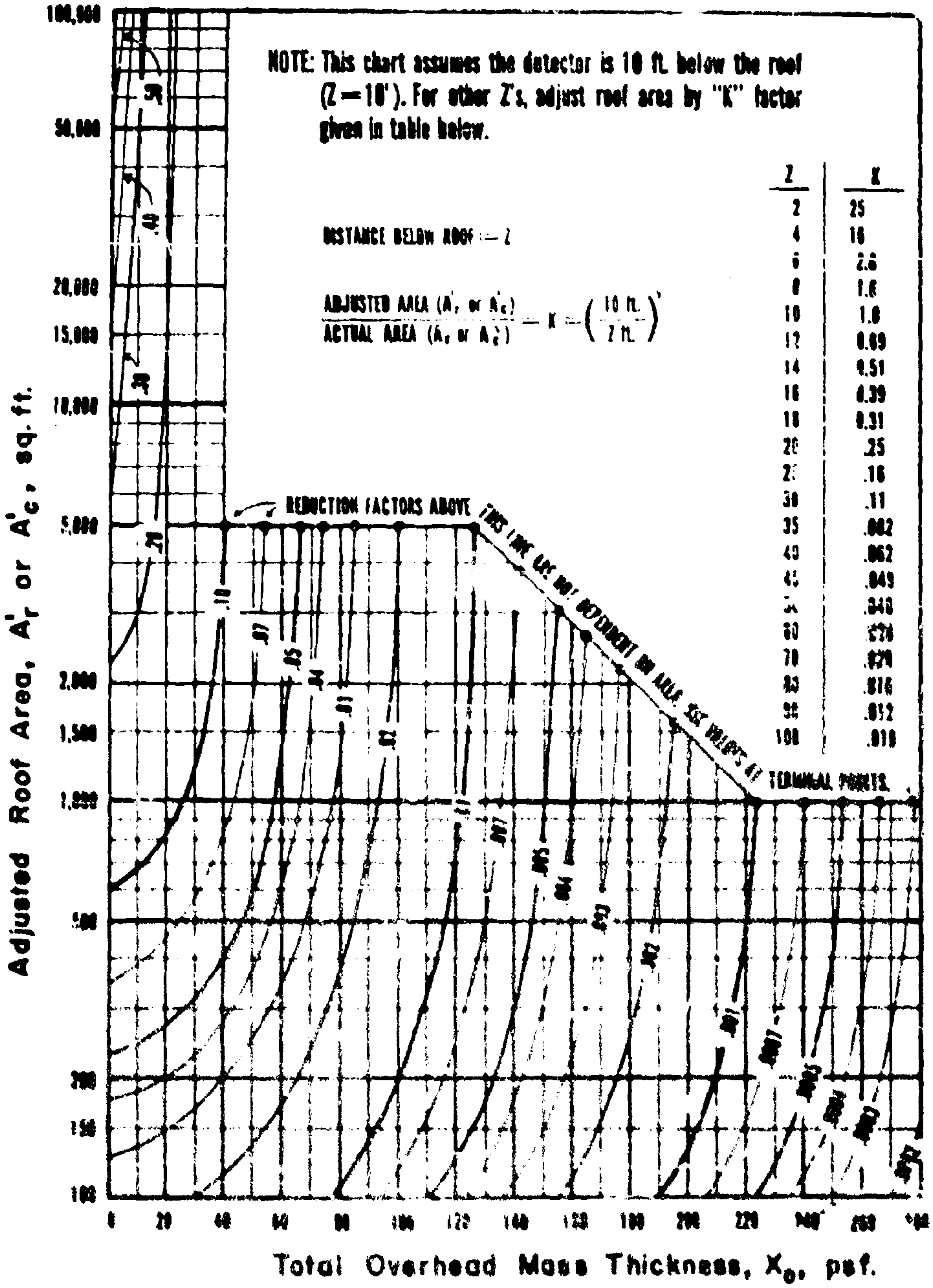


FIG. 5E-1 REDUCTION FACTORS FOR BARRIER SHIELDING EFFECTS, FALLOUT GAMMA RADIATION.



Z	K
2	25
4	16
6	2.8
8	1.6
10	1.0
12	0.69
14	0.51
16	0.39
18	0.31
20	.25
25	.16
30	.11
35	.082
40	.062
45	.049
50	.040
60	.028
70	.020
80	.016
90	.012
100	.010

FIG. 5E-2 REDUCTION FACTORS FOR COMBINED SHIELDING EFFECTS — ROOF CONTRIBUTION

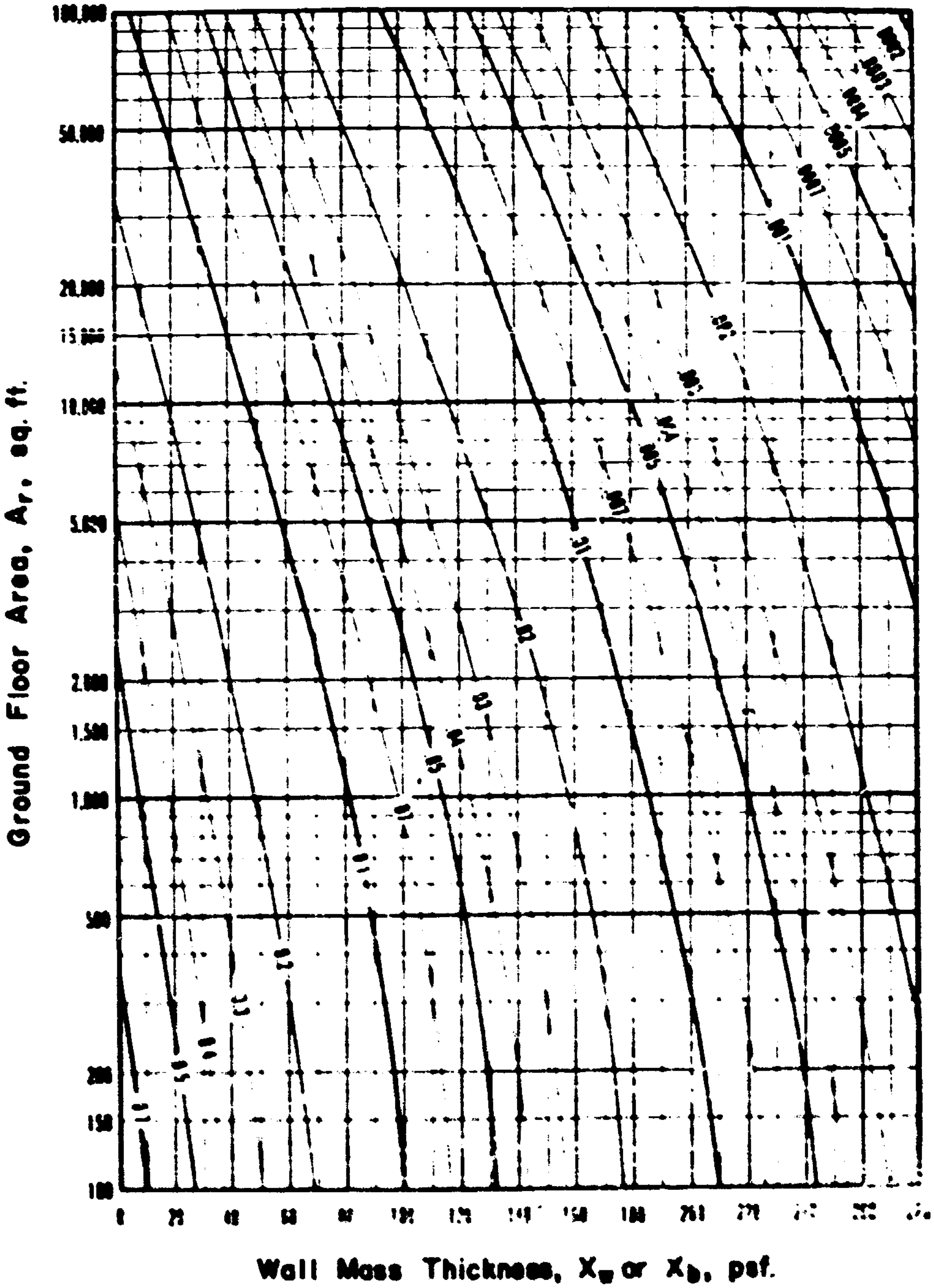


FIG. 5E-3 REDUCTION FACTORS FOR COMBINED SHIELDING EFFECTS —— GROUND CONTRIBUTION —— ABOVEGROUND AREAS

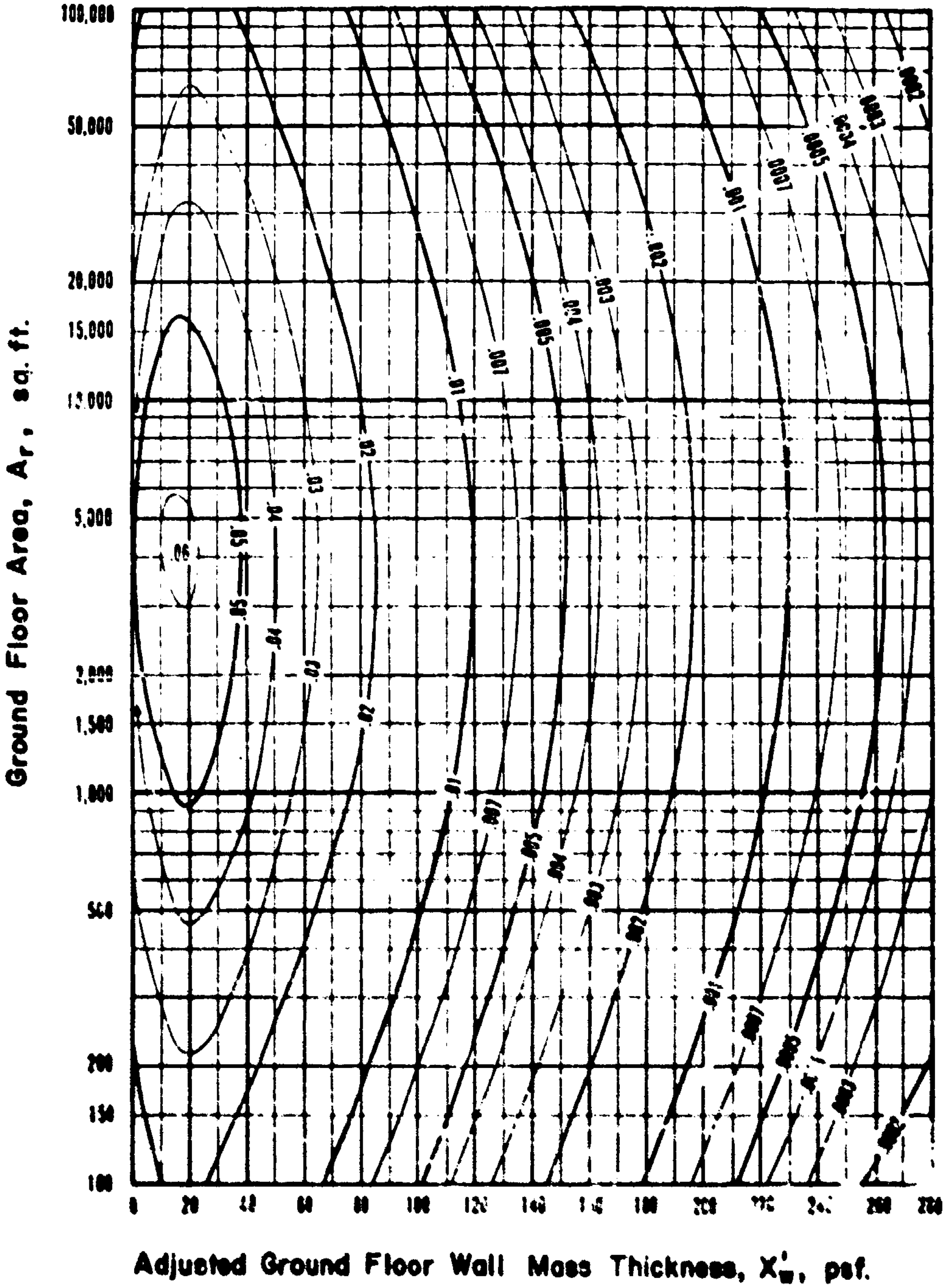


FIG. 5E-4 REDUCTION FACTORS FOR COMBINED SHIELDING EFFECTS — GROUND CONTRIBUTION — BELOWGROUND AREAS.

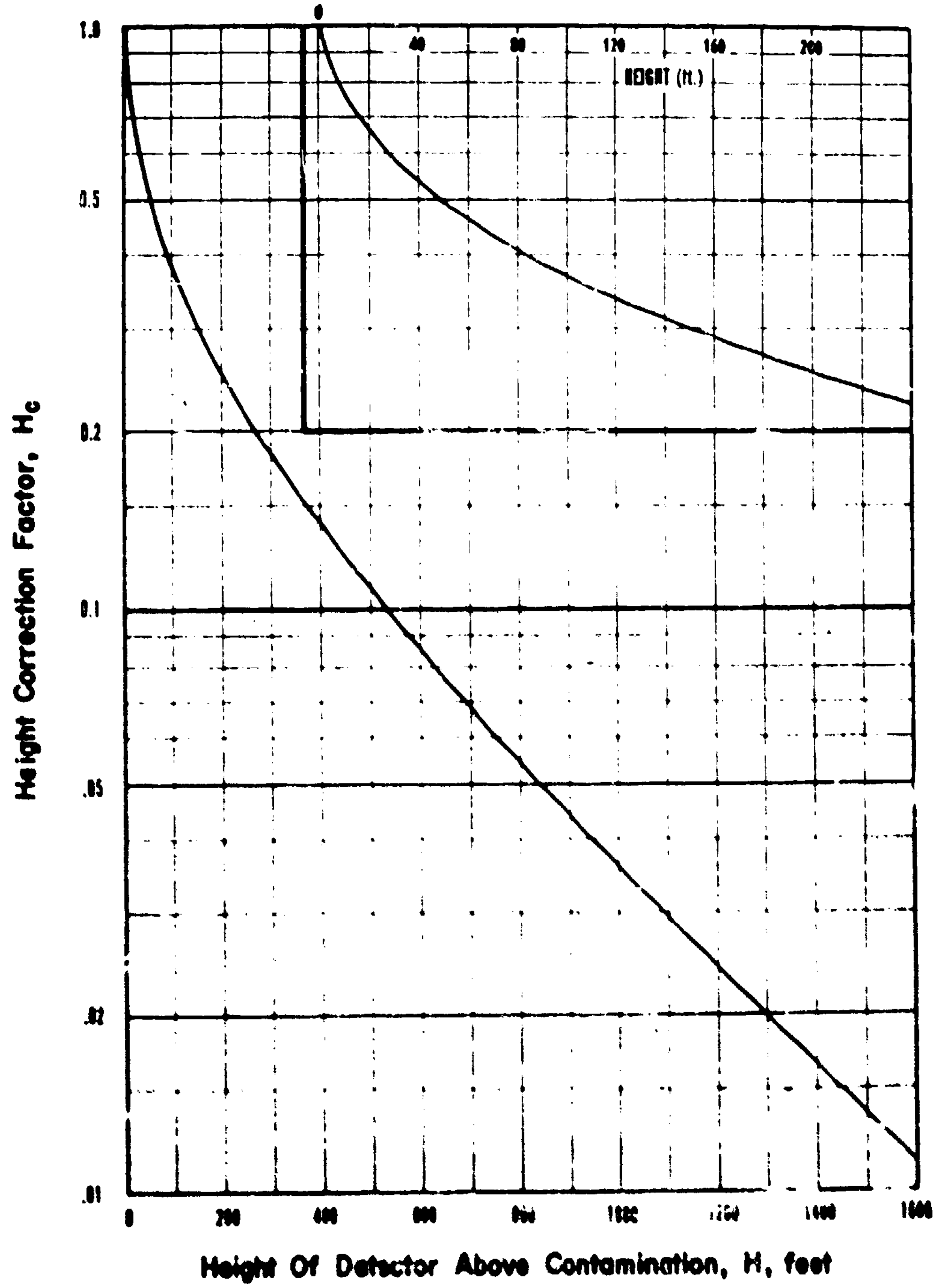


FIG. 5E-5 SHIELDING EFFECTS OF HEIGHTS

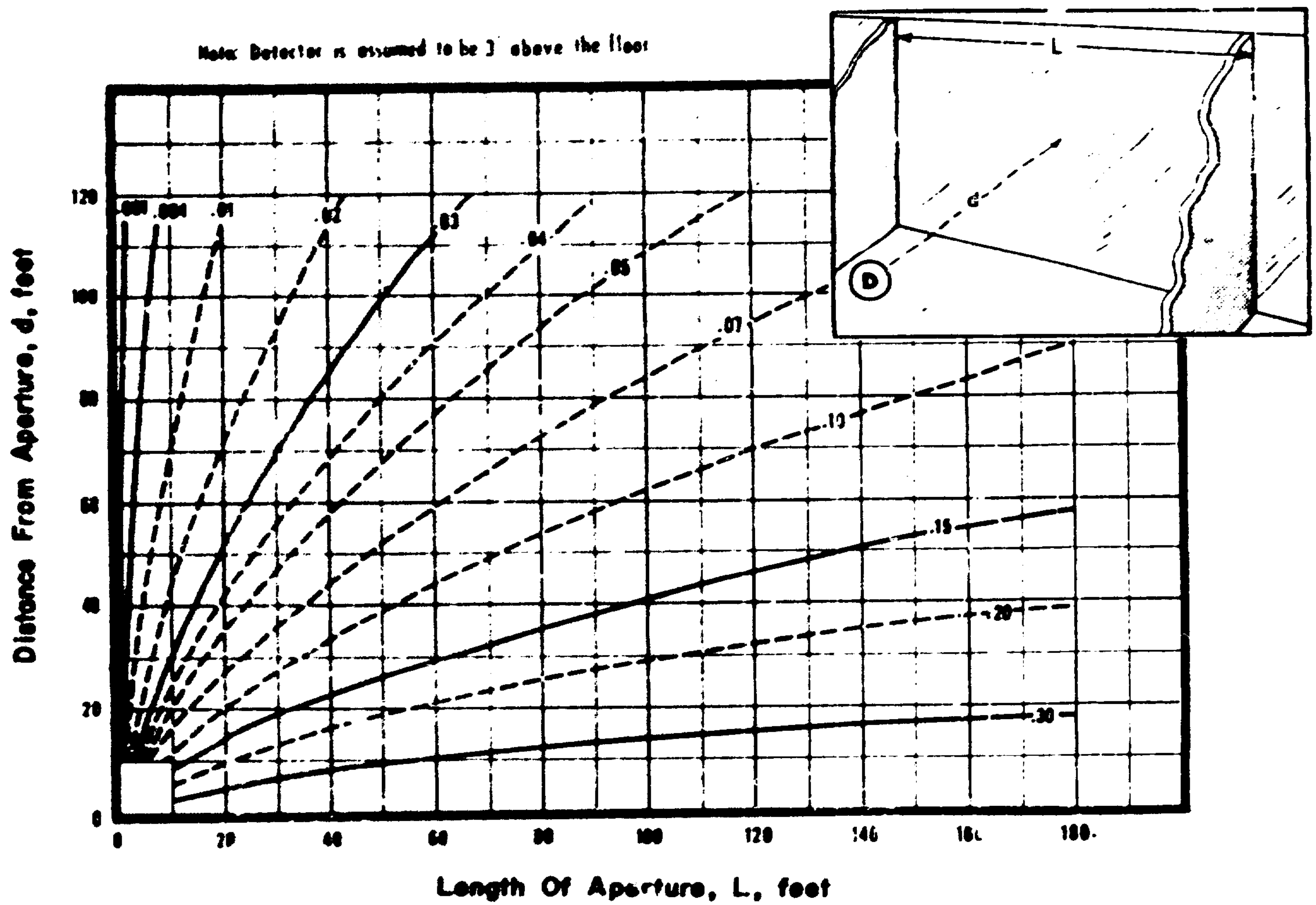


FIG. 5E-6 REDUCTION FACTORS FOR APERTURES GROUND FLOOR.

SECTION 6. ILLUSTRATIVE EXAMPLE

SECTION 6. ILLUSTRATIVE EXAMPLE

6.1 PRESENTATION

This manual is intended primarily as a guide for reviewers of existing or planned hardened construction. In some instances the review process will include (a) determination of levels of resistance (to blast and other effects) inherent in the construction, (b) determination of vulnerability radii corresponding to resistance levels and to weapon sizes of interest, (c) determination of survival probabilities corresponding to the vulnerability radii and assumed C.E.P. values, (d) determination of costs of construction, (e) judgment of the design from the standpoints of all pertinent factors.

At certain stages of the review process it may be more convenient for the reviewer to undertake an independent determination of requirements (functional or physical) and to compare these with the corresponding features of the installation under review. In such instances the reviewer is performing limited, or preliminary design. This manual is applicable to such preliminary design as well as to the reverse process of analysis.

In this section portions of the review process, including certain examples of preliminary design, are illustrated.

6.2 ASSUMED SITUATION

The design of a long-range missile installation consisting of several interdependent structures of essentially equal importance is to be reviewed. In the discussions and analyses that follow attention will be given, primarily, to the control center for this installation. It will be assumed that the control center is located at a considerable distance from other elements and that attacks on these elements will have a negligible effect on the control center survival probability. This simplifying assumption might not be justified in a real case.

The proposed location is one for which no "Blast and Fallout Probability" charts exist. The best available estimate of probable attack indicates three missiles directed at the control center. The warhead is estimated to be 8 MT and the CEP is estimated to be 2 nautical miles. (It is emphasized that these conditions of attack are taken for example purposes only; they may not represent probable conditions in any real situation.) No estimate is available as to the effectiveness of active defenses in the area.

6.3 PROCEDURE

6.3.1 Strategic Category Determination. From SECTION 1, Table 1-1, it is determined that the proposed installation will be in Strategic Category A.

6.3.2 Target Analysis. For the determined Strategic Category what are the desired survival probabilities? For the assumed attack conditions what resistance (to blast, nuclear radiation, and thermal radiation) must be provided in order to achieve the desired survival probabilities?

From Para. 2.3.1 of SECTION 2 it is determined that the desired survival probability consistent with Strategic Category A is in the range 80 - 95 percent. In accordance with the recommendation of Para. 2.4.2b(5) of SECTION 2 the enemy's probability of reaching the target area will be assumed to be 50 percent; i.e., z = 0.50. Since the desired survival probability range is difficult to achieve--particularly for the assumed multiple-shot conditions--required resistances will be determined for the lower end of the range only. Corresponding to a multiple-shot probability of 80 percent, the required single shot probability is given by:

$$_1S_Z = \sqrt[n]{_nS} = \sqrt[3]{0.80} = 0.928$$

The required built-in survival probability (i.e., corresponding to Z = 1.00) is given by

$$_1S_{Z=1} = 1 - \frac{1 - {_1S_z}}{Z} = 1 - \frac{1 - 0.928}{0.5} = 0.856$$

From Fig. 2-4, for S = 0.86 and CEP = 2.0, a value of 5800 ft. is read for the vulnerability radius, R_v. Entering Figs. 2-7, 2-8 and 2-12, with W = 8 MT and R_v = 5800 ft., the corresponding values of side-on overpressure, initial gamma radiation, and thermal radiation are found to be, respectively, 180 psi, 50,000 r, and 4000 cal/cm^2.

In a real case there might be occasion to seek a higher value of survival probability. The required resistance levels would be found to increase very rapidly with increased survival probability, and the installation cost would likewise increase rapidly.

6.3.3 Operational Concepts and Requirements. The total installation might include the control center, power station, guidance and antenna facilities, underground missile launch shelters and their supporting fuel and equipment shelters, the necessary personnel tunnels, fuel, power, and communication conduits, entranceway structures, etc. Specific requirements would be obtained for each of these components. SECTION 3 and other sources would provide data for personnel, space and dimensional needs, ventilation, refrigeration and heating, electrical systems, shock mounting or isolation, type and number of doors and other accessways, and other factors. In an actual case the operational concepts and requirements are complex and warrant extensive study, beyond the scope of this illustration. Specific requirements would be obtained for each component of the installation.

With regard to the control center we will arbitrarily assume an operating personnel of 40 men. The normal space requirement for these men and the necessary equipment is found to be 8000 sq. ft.; on the basis of

Para. 3.3.1 we will assume that this can be reduced to 5000 sq. ft. Other requirements for the control center are of importance, but only the gross space requirement is cited here

6.3.4 Damage Criteria. From SECTION 4 are obtained the vulnerabilities of personnel, structures and equipment to blast, radiation, and shock. For example, from Table 4-1 we determine a value of 100r as the maximum nuclear radiation which should be permitted to penetrate into the control center.

6.3.5 Considerations in the Choice of Structural Type. In judging the suitability of a proposed design, or in undertaking a preliminary design, the reviewer is very much concerned that each structure be appropriate for the resistance levels, site conditions, and operational requirements. If inappropriate structural types are selected either the protective function may not be achieved, or it may be achieved at excessive cost.

Unless the intended function demands aboveground construction, or unless the required resistance levels are quite low, construction belowground will be necessary. The power and accuracy of modern nuclear weapons systems do not offer any practical choice. For example, the side-on overpressures determined in Para. 6.3.2 would generate reflected pressures as great as 1000 psi against an aboveground structure.

The proper type of structure depends, of course, on soil conditions and local topography at the site. On the other hand, the site should be chosen with regard to the total installation requirements. The best site selection based on all factors might not provide ideal conditions for a particular component, such as the example control center.

The cost of underground construction is very sensitive to subsurface conditions, particularly the ground-water level. Where a high ground-water level cannot be avoided economy may dictate shallow construction with a single floor level. Under different circumstances the use of multi-level construction, minimizing plan area, may be less costly.

While the most favorable structural type is not always apparent there are certain considerations which provide guidance. In particular, the choice between shell types and slab (or slab and beam) types is governed by a number of factors. From the standpoint of efficient material use shells are superior because the major stress system can be assumed to be uniform throughout the element thickness. In contrast, slab and beam elements are much less efficient because their major stress states are not uniform throughout the element thicknesses. Slab and beam thicknesses become very large at combinations of loading and span for which shell thicknesses can be relatively small. When the protected function demands large clear spans the shell type may be dictated.

If large clear spans are not essential a slab (or slab and beam) arrangement may be more economical than a shell type. In the first place lower forming costs may be obtained for the plane surfaces of the former types.

Of greater significance may be the relative efficiency of space utilization within the two types. For slab and beam construction interior clear height does not decrease from the interior to the perimeter zones; such height variation is typical of shell types. Unless there are equipment components which can be positioned in the zones of insufficient headroom, these zones will represent wasted space. It may be noted that the shell form which is structurally most efficient (the doubly-curved dome) presents the most challenging problem of space utilization. To a lesser extent this conflict between structural efficiency and space utilization occurs in slab and beam construction. For equal spans and loading the beamless slab requires more material than a slab and beam combination, but the former involves space (overhead, between beams) which may be difficult to utilize.

It should be noted that shell structures for large spans and large loads may present very difficult foundation problems. If rock is not available at reasonably shallow depth, cost may be greatly increased--by the need to go deeper to obtain rock foundation, or by the need for very massive footings. For slab and beam construction the foundation problem is less acute, because the vertical load is transmitted by a large number of columns and walls, and thereby distributed over the entire plan area rather than concentrated at the perimeter.

From the foregoing discussion it should be apparent that the choice of structural type is strongly influenced by loading intensity, but that there are other factors which may be equally significant. Careful study should be given to the operations and equipment to be housed, for these determine minimum acceptable clear spans and the acceptable space configurations. Finally, the availability of good foundation material may be an important factor in the choice of structural type.

6.3.6 <u>Example--Structural Verification of Control Center Using Two-Way Slab Construction</u>. Let us assume that a preliminary design has been submitted for consideration. The reviewer wishes to determine how the resistance of this design compares with the desired levels as determined in Para. 2.3.1.

<u>Description of Structure:</u>

Single story; under 4 ft. of earth cover; 60 ft. x 84 ft. in plan; two-way slab roof system supported on columns spaced 20 ft. in one direction and 21 ft. in the other direction; roof slab 36 in. thick with reinforcement not specified; beam stems 60 in. wide and total beam depth 84 in., with reinforcement not specified; interior columns 48 in. sq. with reinforcement not specified; exterior columns 60 in. x 48 in.; floor slab and beams same as roof; exterior walls 24 in. thick with reinforcement not specified; clear height, from floor to roof beam, 12 ft.; f'_c = 5000 psi for all elements.

The reviewer's task is to determine whether the proposed element thicknesses are appropriate for the resistance levels required; this involves also a determination of reinforcement quantities required. Since the reinforcing steel grade is not specified, intermediate grade steel (the most common grade) will be assumed.

Earth Cover--Adequacy of Radiation Protection:

Checking first for radiation attenuation, we note that the equivalent soil thickness represented by earth cover and roof slab is 4.0 + $\frac{144}{100} \times 3.0 = 8.3$ ft. = 100 in. of soil. From Fig. 5-8 we read 0.0001 for the dose transmission factor. Applying this factor to the initial radiation determined in Para. 6.3.2, we obtain (50,000 r)(0.0001) = 5 r, which is less than the critical value of 100 r found in Para. 6.3.4. Thus, earth cover is adequate.

Roof Slab Panels:

$$\text{Assume } \mu = 3.0 \ ; \ f'_c = 5000 \ ; \ f'_{dc} = 6250 \ ; \ f_{dy} = 52{,}000$$

Dimensions (clear span) = 15.0 ft. x 16.0 ft.

Loading = 180(blast pressure) + 6(dead load) = 186 psi

Pure Shear Strength:

$$\alpha = \frac{15}{16} = 0.94 \ ; \ \frac{d}{L} = \frac{36 - 3}{15 \times 12} = 0.18$$

From Fig. 5A-1.3, $p'_m = 300$ since end steel percentages are taken to be equal.

$$p_m = p'_m(2/3)(1+\alpha) = 300(2/3)(1.94) = 388 \text{ psi} > 186 \quad \text{O.K.}$$

It is concluded that the proposed slab thickness is satisfactory (actually quite conservative) in pure shear.

Flexural Strength: Since the panel is practically square assume equal reinforcement in both directions. Then from Fig. 5A-2.1:

$$\frac{\varphi_{LC} + \varphi_{LE}}{\varphi_{SC} + \varphi_{SE}} = 1.0 \quad \text{and } \Omega = 2.82$$

This means that the flexural capacity of the slab is 2.82 times as large as the capacity of a one-way slab of the same d/L. Let us assume that the d/L found to be adequate for pure shear will likewise be adequate for flexure, and determine the required flexural reinforcement.

$$\text{Required one-way capacity} = \frac{186}{2.82} = 66 \text{ psi}$$

From Fig. 5A-1.2 for d/L = 0.18,

$$(\varphi_{SE} + \varphi_{SE})f_{dy} = 37{,}000$$

$$\text{Therefore } \varphi_{SC} + \varphi_{SE} = \frac{37{,}000}{52{,}000} = 0.71\%$$

Thus, 0.36% steel, each face, each way, will suffice. This is a very moderate steel percentage, and it is concluded that the slab flexural strength is adequate.

__Diagonal Tension Strength__: Again it will be assumed that the d/L is adequate, and we will determine the amount of reinforcement required. The required one-way strength is:

$$p_m' = \frac{p_m}{\frac{2}{3}(1.94)} = \frac{186}{1.29} = 144 \text{ psi}$$

From Fig. 5A-1.8, $\lambda_f = 0.70$

From Fig. 5A-1.7, for $\lambda_f \varphi_c f_c' = (0.70)(0.36)(5000) = 1260$

we obtain $\frac{\lambda_v d}{L} = 0.22$

Thus, $\lambda_v = \frac{0.22}{0.18} = 1.22$

And, from Fig. 5A-1.5 for $\lambda_v = 1.22$, we obtain:

$$\varphi_v f_{dy} = 25{,}000 \text{ psi}$$

$$\text{Required } \varphi_v = \frac{25{,}000}{52{,}000} = 0.48\%$$

By increasing the midspan flexural reinforcement percentage, φ_c, the required diagonal tension reinforcement percentage, φ_v, could be decreased--even to zero. Since the __total__ midspan flexural steel percentage is $4\varphi_c$ (i.e., two ways, two faces) this would not be advantageous. The value of φ_v determined above is satisfactory.

It can be concluded that the proposed roof slab proportions are about right for the desired resistance level, and that the necessary reinforcement percentages are reasonable.

__Roof Beams:__

The beams have clear spans of 17.0 ft. and 16.0 ft. respectively. Since they have identical width and depth, and the spans are almost alike, only the long-span beams need be checked.

__Pure Shear Strength__: Consider, first, an assumed condition for which beams of 16.0 ft. clear span support a one-way slab on a beam spacing, c. to c., of 20.0 ft. The shear capacity of such beams, expressed as load intensity per square inch of slab, can be converted to the corresponding capacity for the actual long-span beams.

Assume $d = 84 - 4 = 80$ in.

Then $\frac{d}{L} = \frac{80}{12 \times 16.0} = 0.417$

From Fig. 5A-3.6 for $d/L = 0.417$ and $f'_c = 5000$ we obtain, since end flexural steel percentages are equal:

$$p'_m \frac{b}{a} = p'_m (20/5) = 710; \quad \text{thus } p'_m = 178 \text{ psi}$$

From Fig. 5A-3.9 for $\alpha = 0.94$, we obtain $\eta = 1.90$

Therefore the capacity of the actual long-span beam, expressed in loading intensity on the slab, is:

$$p_m = \eta\, p'_m = 1.9(178) = 338 \text{ psi} > 186 \quad \text{O.K.}$$

We conclude that the beams are satisfactory (actually conservative) with respect to pure shear.

Flexural Strength: Since the width and depth of the beam were found to be satisfactory, the flexural check will be a determination of the required reinforcement.

From Fig. 5A-3.5 for $\alpha = 0.94$ and $a/b = 5/20 = 0.25$, we read

$$\gamma' = 1.28$$

The required capacity for a beam of 16.0 ft. span supporting a one-way slab with 20.0 ft. beam spacing is:

$$\frac{186}{1.28} = 145 \text{ psi}$$

and $p'_m \frac{b}{a} = 145(20/5) = 580$ psi

From Fig. 5A-3.2 for $d/L = 0.417$ and $p'_m \frac{b}{a} = 580$, we obtain

$$(\varphi_c + \varphi_e) f_{dy} = 55{,}500 \text{ psi}$$

$$\varphi_c + \varphi_e = \frac{55{,}500}{52{,}000} = 1.07\%$$

Thus $\varphi_c = \varphi_e = 0.54\%$. This percentage of flexural reinforcement (0.54% at ends and at midspan) is satisfactory.

Diagonal Tension Strength: Again it will be assumed that the d/L is adequate, and the required amount of reinforcement will be determined.

From Fig. 5A-3.9 for $\alpha = 0.94$, we obtain $\eta = 1.9$

The required capacity of a beam of 16.0-ft. span supporting a one-way slab on 20.0-ft. beam spacing would be:

$$p_m' = \frac{p_m}{\eta} = \frac{186}{1.9} = 98 \text{ psi}$$

and

$$p_m' \frac{b}{a} = 98(20/5) = 392 \text{ psi}$$

From Fig. 5A-1.8, $\lambda_f = 0.70$

Therefore $\lambda_f \varphi_c f_c' = (0.70)(0.54)(5000) = 1890$, and

from Fig. 5A-3.8 for $p_m' \frac{b}{a} = 392$, we obtain:

$$\frac{\lambda_v d}{L} = 0.34$$

Thus $\lambda_v = \frac{0.34}{0.417} = 0.82 < 1.0$

Since λ_v is less than 1.0, no diagonal tension reinforcement is theoretically required. Nevertheless, it is recommended that a minimum percentage, $\varphi_v = 0.25$ be used.

It can be concluded that the proposed roof beams are about right for the desired resistance level, and that the necessary reinforcement percentages are reasonable.

Interior Columns:

The check of these members involves a determination of the required reinforcement percentage.

From Para. 5A-3.10, the column capacity should be equal to twice the blast loading on one tributary roof area, or to the strength of the roof elements, whichever is smaller. The ductility ratio, μ, was taken as 3.0 for the roof elements; this means that the strengths were made equal to the load intensity multiplied by:

$$\frac{2\mu}{2\mu - 1} = \frac{2(3.0)}{2(3.0)-1} = \frac{6}{5} = 1.2 < 2.0$$

Thus the column strengths must be made equal to:

$$1.2[(21x12)(20x12)(0.186)] = 13,500 \text{ kips}$$

From Para. 5B.11, the column capacity is given by:

$$P_u = (0.85 f_{dc}' + \frac{\varphi_T}{100} f_{dy}) mn$$
$$= (5312 + 520 \varphi_T)(48x48)$$

$$\text{Thus } \varphi_T = \frac{1}{520}\left[\frac{13,500,000}{48 \times 48} - 5312\right] = \varphi_T = 1.05\%$$

The required reinforcement percentage is satisfactory. By increasing the steel content the column dimensions could be reduced somewhat. To achieve any substantial reduction in column size, however, it would be necessary to substitute a steel column for reinforced concrete.

It should be noted that even the 48 in. x 48 in. column proposed would require a small capital enlargement to the 60-in. width of the supported beams.

Exterior Walls:

In accordance with recommendations in Para. 5.3.3, and assuming unsaturated cohesive soil of medium consistency, the lateral soil pressure will be taken equal to half the vertical pressure; i.e., 90 psi. In view of the shallow depths considered, the attenuation of blast pressure with depth, as well as the static lateral pressure, can be neglected.

The roof slab and beams were designed as continuous. Therefore, at the connection between the side walls and roof, the moment resistance required for the roof elements must be supplied in the corresponding wall elements.

The side walls and columns must be adequate to resist the direct compressive loads equal to the reactions from the roof plus the moments which result from continuity of the roof system and the laterally applied load.

In view of the assumptions that floor system and roof system are alike the wall panels can be considered as restrained on all four edges. For the assumed exterior column dimensions, the wall panels on the long sides of the structure have horizontal clear spans of 16.0 ft. Vertical clear spans also are 16.0 ft. Square panels.

Pure Shear Strength:

Assume $d = 24$ in. $- 4$ in. $= 20$ in.

$$\frac{d}{L} = \frac{20}{16 \times 12} = 0.104$$

$$\alpha = 1.0$$

From Fig. 5A-1.3, $p_m' = 160$ psi for equal end reinforcement.

$$p_m = p_m'(2/3)(1+\alpha) = (160)(2/3)(2.0) = 213 \text{ psi} > 90$$

We conclude that the 24-in. thickness of the wall slabs is more than adequate with respect to pure shear strength. It would be premature to conclude that the thickness is excessive until flexure and diagonal tension have been investigated. It should be noted that the preceding computation ignores the

effect of non-uniformity of flexural steel percentages in the two directions on the maximum edge shears and may, therefore, be somewhat unconservative. However, it also neglects the effect of the direct vertical compression which should serve to increase the shear strength of the wall above that which was used here. In any case, the resistance as computed (213 psi) is so much larger than the load (90 psi) that safety is assured.

Flexural Strength: Moment capacity of roof slab is given by:

$$M = 0.009\phi f_{dy} d^2 = 0.009(0.36)(52,000)(33)^2 = 183,500 \text{ in.-lb./in.}$$

Neglecting the effect of direct compression in the wall, the vertical steel at top and bottom edges of the wall panel required to develop the end moment in the roof slab is:

$$\phi_e = \frac{183,500}{(0.009)(52,000)(20)^2} = 0.98\%$$

Near the midlength of upper and lower panel edges, there is no doubt that this is very (perhaps unnecessarily) conservative. This is shown as follows:

In direct compression, the ultimate strength of the wall is given by:

$$P_u = (0.85\, f'_{dc} + \frac{\phi_T}{100} f_{dy})(24 \times 1) \text{ lbs./in.}$$

Taking ϕ_T as approximately 1.5 (0.98% on one side and an estimated 0.50% on the other):

$$P_u = [(0.85)(6250) + (1.5)(520)]\ (24 \times 1)$$
$$= (5320 + 780)(24) = 144,650 \text{ lbs./in.}$$

Near the midlength of upper and lower panel edges:

$$P \cong [(186)(10' \times 12''/')] \div [(2/3)(1 + \alpha)]$$

where $\alpha = 14/15 = 0.94$ (for roof panel):

$$P \cong 22,300/1.29 = 17,300 \text{ lbs./in.}$$

Thus: $\frac{P}{P_u} = \frac{17,300}{144,650} = 0.12$

From Fig. 5A-10.2, for $\phi = 0.98$, $f'_c = 5000$, and $P/P_u = 0.12$,

$$\frac{M}{M_u} = 1.7$$

and the moment capacity of the wall consistent with 0.98% end steel is about

70% greater than is needed. However, the intensity of the direct compression is reduced as the edge of the panel is approached and the corresponding excess moment capacity is reduced. The average direct compression in the wall panel is approximately half of the "simple span" roof slab shear, or:

$$P_{avg} \cong \frac{1}{2} [(186)(10' \times 12"/')] = 11,150 \text{ lbs./in.}$$

For this

$$\frac{P_{avg}}{P_u} = \frac{11,150}{144,650} = 0.077$$

and $\frac{M}{M_u} = 1.4$;

therefore the average excess resistance is about 40%.

On the basis of the above discussion, it is reasonable to reduce the top and bottom wall steel percentages from 0.98% to about 0.70%. Checking this:

$$P_u \cong [(0.85)(6250) + (0.70 + 0.50)(520)] (24 \times 1)$$

$$\cong (5320 + 625)(24) = (5945)(24) = 142,500 \text{ lbs./in.}$$

$$\frac{P_{avg}}{P_u} = \frac{11,150}{142,500} = 0.078$$

From Fig. 5A-10.2, for $\varphi = 0.70$, $f'_c = 5000$, and $\frac{P}{P_u} = 0.078$

$$\frac{M}{M_u} = 1.4$$

$$M_u = 0.009\ \varphi f_{dy} d^2$$

$$= 0.009(0.70)(52,000)(20)^2 = 131,000 \text{ in. kips/in.}$$

Then $M = 1.4(131,000) = 183,500$ in. kips/in., which is equal to the moment resistance required by the roof slab.

Therefore use 0.70% vertical steel in the outer face at top and bottom edges of wall panels. Nominal steel of about 0.50% will also be required in the inside face, at top and bottom edges of the panel.

In order to maximize the two-way action of the panel assume $\varphi_{LC} + \varphi_{LE} = \varphi_{SC} + \varphi_{SE}$. Then from Fig. 5A-2.1, Ω is found to be 3.0. The one-way capacity in each direction must then be equal to $90/3 = 30$ psi.

From Fig. 5A-1.2 for $p_m = 30.0$ and $d/L = 0.104$, we obtain:

$$(\varphi_c + \varphi_e) f_{dy} = 46{,}500$$

$$\text{and} \quad (\varphi_c + \varphi_s) = \frac{46{,}500}{52{,}000} = 0.90\%$$

Obviously with 0.70% required at top and bottom, the $(\varphi_C + \varphi_E)$ for vertical steel will exceed 0.90% since $\varphi_C = 0.90 - 0.70 = 0.20\%$ is less than the nominal amount permitted when steel is required. Designating the vertical span as the "short" span, let us assume:

$$\varphi_{SE} = 0.70$$

$$\varphi_{SC} = \varphi_{LE} = \varphi_{LC} = 0.50$$

Then

$$\frac{\varphi_{LC} + \varphi_{LE}}{\varphi_{SC} + \varphi_{SE}} = \frac{1.0}{1.20} = 0.833$$

From Fig. 5A-2.1, $\Omega = 2.75$

The one-way capacity in the vertical direction then must be $\frac{90}{2.75} = 33$ psi

From Fig. 5A-1.2, for $p_r = 33.0$ and $d/L = 0.104$, we obtain,

$$(\varphi_{SC} + \varphi_{SE}) f_{dy} = 51{,}000$$

Thus required $(\varphi_{SE} + \varphi_{SE}) = \frac{51{,}000}{52{,}000} = 0.98 < 1.20$ O.K.

We conclude that the indicated percentages of flexural reinforcement are satisfactory.

Diagonal Tension Strength:

From Fig. 5A-1.8, $\lambda_r = 0.70$

Then from the preceding section, $\varphi_c = 0.5$

$$\lambda_r \varphi_c f'_c = (0.70)(0.5)(5000) = 1750$$

For $\alpha = 1.0$, the equivalent shear capacity for one-way slab action is:

$$p'_m = \frac{p_m}{\frac{2}{3}(1 + \alpha)} = \frac{90}{\frac{2}{3}(2.0)} = 67.5 \text{ psi}$$

From Fig. 5A-1.7, for $p'_m = 67.5$ and $\lambda_r \varphi_c f'_c = 1750$, we obtain:

$$\lambda_v \frac{d}{L} = 0.14$$

Thus $\lambda_v = \frac{0.14}{0.104} = 1.35$

From Fig. 5A-1.5, for $\lambda_v = 1.35$, we obtain:

$$\varphi_v f_{dy} = 42,000$$
$$\varphi_v = \frac{42,000}{52,000} = 0.81\%$$

We conclude that the proposed thickness for the exterior wall is adequate, and the required reinforcement percentages are not excessive.

Exterior Columns:

Proposed width and depth are 60 in. and 48 in. respectively.

Pure Shear Strength:

Assume d = 48 - 4 = 44 in.

$$\frac{d}{L} = \frac{44}{12 \times 12} = 0.31$$
$$\alpha = 1.0$$

From Fig. 5A-3.6, $p_m \frac{b}{a} = 1050$, since φ_e at each end may be taken as equal.

Thus $p_m = 1050(a/b) = 1050(5/21) = 250 \text{ psi} > 90$ O.K.

We conclude that the columns are conservative with respect to pure shear strength.

Flexural Strength. Since the top and bottom moment capacities must each equal the end moment capacity of the roof beams, we first determine the reinforcement percentages for the top and bottom exterior steel.

A wall column can be assumed to receive as a direct compression load a force equal to the end reaction of the roof beam which frames into it. For the case being treated, this is:

$$P = \frac{1}{2}\,[(20 \times 12)(10 \times 12)]\,(186)$$
$$= 2,680,000 \text{ lbs.}$$

Assuming a total steel percentage in the column of 1.5%:

$$P_u = [(0.85)(6250) + (1.5)(520)]\,(60 \times 48)$$
$$= (5320 + 780)(2880)$$
$$= (6100)(2880) = 17,580,000 \text{ lbs.}$$

and $\frac{P}{P_u} = \frac{2.68}{17.6} = 0.152$

Then, from Fig. 5A-10.2 for $f'_c = 5000$ and assuming $\varphi = 1.5\%$:

$$\frac{M}{M_u} \cong 1.6$$

Thus, we may (assuming our assumptions thus far to have been reasonable) proportion the flexural steel in the column for a moment equal to 1/1.6 times the end moment capacity of the roof beam. Thus:

For roof beam: $d = 80.0$ in.; $\varphi_E = 0.54\%$

For Wall column: $d = 44.0$ in.

For equal moment capacities:

$$\varphi_E \text{ (for column)} = \left(\frac{80}{44}\right)^2 (0.54) = 1.78\%$$

For required column moment:

$$\varphi_E = (1/1.6)(1.78) = 1.11\%$$

which is less than the 1.5% assumed.

Revising the computations:

From Fig. 5A-10.2, for $f'_c = 5000$, $P/P_u = 0.157$, and assuming $\varphi = 1.0\%$:

$$\frac{M}{M_u} \cong 1.8$$

To obtain required column moment:

$$\varphi_E = (1/1.8)(1.78) = 0.99\% \cong 1.0\% \text{ as assumed.}$$

Accept this as being reasonable end reinforcement percentage. It also agrees with the original assumption of a total steel percentage (for computation of P_u) of 1.5% since a nominal amount of about 0.5% will be required on the compression face (inside) at the ends of the column.

To obtain the steel required at the center of the column,

refer to Fig. 5A-3.4 where for $d/L = 0.31$

and $p_m\left(\frac{b}{a} + \frac{1}{2}\right) = 90\left(\frac{21}{5} + \frac{1}{2}\right) = 423$,

$(\varphi_c + \varphi_e)f_{dy} = 49{,}000$ psi

Thus: $\varphi_c + \varphi_e = \frac{49,000}{52,000} = 0.94\%$

and $\varphi_e = 0.94 - \varphi_c = 0.94 - 1.0 = -0.06\%$

Therefore theoretically, no interior steel is required at the column center, but a nominal amount ($> 0.06\%$) is needed on the outside of the column. To insure ductility, require 0.5% on both faces of the column.

Diagonal Tension Strength:

From Fig. 5A-1.8, $\lambda_f = 0.70$

Then $\lambda_f \varphi_c f'_c = (0.70)(0.5)(5000) = 1750$

$$p_m \frac{b}{a} = 90(21/5) = 378 \text{ psi}$$

From Fig. 5A-3.8 for $\lambda_f \varphi_c f'_c = 1750$ and $p_m \frac{b}{a} = 378$, we obtain:

$$\frac{\lambda_v d}{L} = 0.23$$

and

$$\lambda_v = \frac{0.23}{0.31} = 0.74$$

Therefore, since $\lambda_v < 1.0$, no web steel is required.

We conclude that the exterior column dimensions are satisfactory, and the reinforcement percentages are reasonable.

Floor System:

Since floor slab and floor beams were stated to be similar to roof slab and beams, and because the (upward) floor system loading in only slightly greater than the roof loading, no review of the floor system is required.

Other Considerations:

In an actual design, careful attention must be given to a number of structural factors which are beyond the scope of this illustrative treatment. It might be advantageous to locate the roof beams and exterior columns outside the roof and wall slabs, for example. Such an arrangement would permit a few feet reduction in total construction depth. Horizontal planes through the beam stems (and vertical planes through the column stems) would be subjected to tensile stress and might require additional reinforcement. It may be noted that this condition exists in the floor beams as proposed.

Because of the large d/L values of the floor beams, their distortions will be quite small, even though a μ value of 3.0 was assumed. Nevertheless the effect of such distortion on equipment, and indeed, the

whole problem of isolating personnel and equipment from shock effects would require careful study.

Finally, extreme care would be required to avoid the introduction of local structural weaknesses at abrupt section changes required for joints, mountings, holes for tunnels and conduits, entrances, etc.

6.3.7 Preliminary Design of Alternate, Flat Slab Roof for Control Center. For the two-way slab design reviewed above the depth of floor and roof beams had to be relatively large. If it is necessary or desirable to minimize the construction depth the reviewer might wish to explore the possibility of using flat slabs for floor and roof. As was noted in Para. 6.3.5, flat slabs require more material than slab and beam construction, but they are more efficient from the standpoint of space utilization.

Let us assume a flat slab roof of 54.0 in. total depth, without drop panels, and with column capitals approximately 6.0 ft. square. If the interior clear height is maintained at 12.0 ft., the total structure height would be 21.0 ft.; this is 5.0 ft. less than the total height of the previously studied slab and beam type structure. Because of the greater roof slab thickness no earth cover would be required for radiation attenuation, and the cover could be the minimum deemed necessary for concealment. Hence, the required depth of construction can be reduced substantially if the flat slab scheme is feasible. The criterion will be the magnitude of the reinforcement percentages required.

The assumption that drop panels will not be used may require some explanation. Assuming that the interior clear height must be maintained over the entire area outside the capitals, and that we hope to limit the maximum slab thickness to 54.0 in., a drop panel would imply thicknesses less than 54.0 in. outside the drop panel zones. Since the total flexural capacity thereby would be reduced, drop panel construction will not be assumed.

Since two layers of reinforcement probably will be needed in each direction, assume:

$$d = 54.0 - 4.0 = 50.0 \text{ in.}$$

$$\frac{d_p}{L_1} = \frac{d}{L_1} = \frac{50}{12\text{x}21} = 0.2$$

$$\frac{c}{L_1} = 0.3 \quad ; \quad \frac{L_2}{L_1} = \frac{20}{21} = 0.95$$

From Fig. 5A-5.6, $K_1 = 1.48$

$$\frac{d_p}{L_1 K_1} = \frac{0.2}{1.48} = 0.135$$

From the expressions on Fig. 5A-5.5 for $d_p/L_1K_1 = 0.135$ and $p_{mv} = 186$, we obtain $f'_c\, p_{mf} - 3830 \times 10^3$

Thus the required flexural capacity is:

$$p_{mf} = \frac{3830 \times 10^3}{5000} = 766 \text{ psi}$$

From Fig. 5A-5.4, for $C_1/L_1 = C_2/L_2 = 0.3$ and $L_2/L_1 = 0.95$, we obtain K = 0.95

$$p_{mf}\, K = (766)\, 0.95 = 728 \text{ psi}$$

From the expression on Fig. 5A-5.2 for $p_{mf}\, K = 728$ and $d/L_1 = 0.2$, we obtain $f_{dy}\varphi = 303{,}000$

Thus required $\varphi = \frac{303{,}000}{52{,}000} = 5.83\%$

From Fig. 5A-5.3 for $d_p/d = 1.0$ and $p_1/L_1 = 0$, we obtain

$X = 1.0$

Now assume that the reinforcement percentages are everywhere equal; i.e., $\varphi_{b_1} = \varphi_{b_2} = \varphi_{t_1} = \varphi_{t_2} = \varphi_o$

Then $\varphi = \varphi_o + \varphi_o + 1.0(\varphi_o + \varphi_o) = 4\varphi_o$

and $\varphi_o = \varphi/4 = 5.83/4 = 1.46\%$ O.K.

We conclude that flat slab construction with a total slab thickness of 54.0 in. is feasible. The required reinforcement percentages are large, but the implied steel could be supplied by two layers of steel in each direction.

6.3.8 Preliminary Designs of Alternate Reinforced Concrete Barrel Arch Roofs for Control Center. To achieve the advantages of large clear spans it is necessary to use a shell type roof. As was noted in Para. 6.3.5, this type may require high-capacity soil, or rock, at reasonable depth because of concentrated reactions to be resisted at the springing line. For the purposes of this illustration let us assume that foundation conditions are suitable. Let us further assume that 5000 sq. ft. of space. unimpeded by columns, and with a minimum height of 6.0 ft. are desired.

Major Dimensions:

An infinite number of combinations of central angle, radius, and barrel length would satisfy the above-stated space requirements. As radius is reduced the shell thickness decreases, but the maximum height of structure (at crown) increases for a given transverse span. Obviously the acceptable planform is significant; i.e., will a working space 50.0 ft. x 100.0 ft. satisfy, or must the smaller clear span be at least 65.0 ft.? For purposes of

this example let us assume that a minimum working space dimension of 62.5 ft. is desired; the length of the barrel then must be 80.0 ft. to supply the required 5000 sq. ft. of working area.

Try a minimum central angle of 120 degrees between springing points. If r is the radius to the shell mid-plane and the thickness, D, is estimated to be about 2.0 ft., the rise of the intrados curve will be:

$$(r - 1)(1 - \cos 60^{o}) = 0.5(r - 1)$$

Then r can be determined from the requirement of a 62.5-ft. transverse dimension with 6.0 ft. headroom.

$$[0.5(r-1) - 6.0][2(r-1) - 0.5(r-1) + 6.0] = \frac{(62.5)^2}{4}$$

From which $r = 42$ ft.

The total depth of the extrados curve (at crown) will be approximately 21.5 ft., and the transverse span of the extrados curve (at springing) will be about 76 ft. If a smaller central angle were chosen the maximum height would increase and the transverse span would decrease. For this example the above values of radius and central angle are assumed to be satisfactory.

Earth Cover:

For purposes of radiation attenuation a depth of cover at the crown, H_c, of 3.0 ft. will suffice. To achieve the advantages of structural behavior associated with "full burial" (see Para. 5A-3.7) H_c must be at least 0.125 times the transverse span, B.

$$B = 42(1.732) = 72.8 \text{ ft.}$$

$$0.125\, B = 9.1 \text{ ft.}$$

In addition, to be considered "fully buried" the average cover, H_{av}, must be at least 0.25 B. For the arch here considered the latter requirement governs; that is, an average cover of 0.25 B requires a crown cover $H_c = 12.4$ ft., which is greater then 0.125B.

Required arch thickness and reinforcement will be determined for each of two cases; i.e., $H_c = 3.0$ ft. and $H_c = 12.4$ ft.

"Partial Burial" Case ($H_c = 3.0$ ft.):

From Fig. 5A-7.5, we can read several combinations of acceptable Φf_{dy} and d/r $1/p_{so}$. However the steep slope of the curves indicates that the arch thickness is not sensitive to changes in percentage reinforcement. Assuming Φf_{dy} equal to 20,000, and interpolating for $f'_{dc} = 6250$, we find:

$$\frac{d}{r}\frac{1}{p_{so}} = 0.000225$$

Thus $d = (42x12)(180)(0.000225) = 20.4$ in.

$$\varphi = \frac{20,000}{52,000} = 0.38\% \quad \text{O.K.}$$

We must check the arch capacity with respect to buckling. From Para. 5A-3.7, the critical load as governed by buckling is:

$$p_{CR} = 1.5(1 - \frac{\beta^2}{\pi^2})(\frac{\pi}{\beta})^2 \frac{E}{18} (\frac{D}{r})^3$$

$$= 1.5(1 - \frac{1}{9})(9)(\frac{3.6x10^6}{18})(\frac{2}{43})^3 = 242 > 180$$

We conclude that a total shell thickness of 24.0 in. with 0.38% reinforcement in each face will be sufficient.

"Full Burial" Case ($H_c = 12.4$ ft.):

$$H_{av} = 0.25\ B = 0.25(72.8) = 18.2 \text{ ft.}$$

Estimating the total shell thickness, D, at 1.8 ft., we obtain:

$$H_{av} + D = 18.2 + 1.8 = 20.0 \text{ ft.}$$

Assuming a reinforcement percentage $\varphi_T = 0.75\%$ (i.e., 0.37% in each face) the unit strength of the shell section is:

$$0.85f'_{dc} + 0.009f_{dy}\varphi_T = 0.85(6250) + 0.009(52,000)(0.75)$$

$$= 5310 + 350 = 5660 \text{ psi}$$

From Fig. 5A-7.1 we obtain $(\frac{D}{r})_{H_{av}+D} = 0.003$

From Fig. 5A-7.3 for $p_m = 180$, we read $(\frac{D}{r})_{p_m} = 0.039$

Thus $\frac{D}{r} = 0.003 + 0.039 = 0.042$

And required $D = 0.042(42x12) = 21$ in.

We conclude that a total shell thickness of 21.0 in., with 0.37% reinforcement in each face will suffice. It is of interest to note that the required thickness is essentially the same for the "fully-buried" case as for the "partially-buried" case.

The ends of the barrel shell structure can be closed off either by a slab or by a double-curved shell. If the latter is chosen it will be found

to require a shell thickness approximately one-half as large as that of the barrel shell. If the doubly-curved shell is used for end closure the length of the barrel shell portion can, of course, be reduced somewhat.

SECTION 7. ACKNOWLEDGMENT AND REFERENCES

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7.1 ACKNOWLEDGMENT

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The curves shown in Figs. 5-21 and 5-23 for shock attenuation in tunnels and ducts are based on data made available for this use by W. J. Taylor and R. O. Clark of the Ballistic Research Laboratories, Aberdeen Proving Ground, Maryland. In addition, Fig. 5-22 is based on an expression developed by Mr. Clark.

7.2 REFERENCES

The following are referred to in the text of the report. Other references of a much less general nature are listed in APPENDICES 5C and 5D, and in Paras. 5.2, 5.7, 5.8 and 5.9.

1. "The Effects of Nuclear Weapons", Department of Defense, D/A No. 39-3, May 1957 (Uncl.).

2. "Capabilities of Atomic Weapons", TM23-200, 1957 (Confidential).

3. "Radiological Recovery of Fixed Military Installations", Departments of the Army and Navy, TM-3-225, NAVDOCKS TP-PL-13, 16 April 1958. Interim Revision. (Uncl.)

4. Terzaghi, K., "Theoretical Soil Mechanics", John Wiley and Sons, Inc., New York, 1943, see pp. 194-215.

5. "Considerations in the Design of Underground Protective Structures", by Nathan M. Newmark, Vol. III, Final Report of Special Studies Related to the WS 107A-2 Launcher Installation, American Machine and Foundry Co., Contract AF-04(647)-156, 20 June 1958 (Secret).

6. "Shock Effects on Equipment", Vol. IV, Final Report of Special Studies Related to the WS 107A-2 Launcher Installation, American Machine and Foundry Co., Contract AF-04(647)-156, 15 Aug. 1958 (Secret).

7. Office of Civil and Defense Mobilization, "Fallout Shelter Surveys: Guide for Architects and Engineers," NP-10-2, May 1960 (Uncl.).

8. "Ground Shock, Structural Response, and Thermal Radiation", Vol. II, Final Report of Special Studies Related to the WS 107A-2 Launcher Installation, American Machine and Foundry Co., Contract AF-04(647)-156, 6 Aug. 1958 (Secret).

9. Elswick, W. R., "Thermal Problems", Proceedings of the Symposium on Protective Construction, ASTIA Document No. AD 150 659, the Rand Corp. (Secret, Formerly Restricted Data).

10. "Engineering Study of Atomic Blast Resistant Design for Several Building Types", by Ammann and Whitney, Final Report, Contract DA49-129-eng-317, March 1960 (Uncl.).

11. Curione, C., "Cost Guidance for Protective Construction", Technical Study No. 24, Department of the Navy, Bureau of Yards and Docks, 20 August 1958 (Uncl.).

12. Koike, R. S., Morrison, T. G., and Tuggle, W., "Cost Planning Information", American Machine and Foundry Co., Contract AF 33(616)-2457 with Air Force Special Weapons Center, ARDC, October 1956 (Confidential).

13. "A Protective Alert Shelter for Strategic Air Command", Associated Research Design, Contract AF 18(600)-1636 with Air Force Special Weapons Center, ARDC, 1 April 1957 (Secret, Restricted Data).

14. "USAF Pricing Guide for Permanent Construction in Continental U.S.", Directorate of Installations DSC Operations, HDq. USAF, October 1958 (Uncl.).

15. Newmark, N. M. and Hall, W. J., "Preliminary Design Methods for Underground Protective Structures", AFSWC-TF-60-5, Contract AF 29(601)-1171, Dec. 1959 (Secret).

16. "Nuclear Radiation Criteria for Hardened ICBM Systems", Space Technology Laboratory, Report STL/TR-59-0000-00735, Dec. 1959 (Secret-RD)

17. Brode, H. L., "Weapons Effects for Protective Design Ground Support Systems Weapons Effects", Rand Corp. Report P-1951, March 31, 1960 (Uncl.).

18. White, Clayton S., "Biological Blast Effects", Lovelace Foundation for Medical Education and Research, AEC Contract No. AT (29-1)-1242, Sept. 1959 (Uncl.).